Thomas W. Kennedy

HIGHWAY RESEARCH RECORD

Number 351

Soil Stabilization: Asphalt, Lime, Cement

10 Reports

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FOREWORD

The dwindling supplies of natural resources for road building are being further curtailed in use by zoning regulations and environmental considerations. This fact gives greater emphasis to the need for improving the utilization of marginal and submarginal materials through modification and stabilization techniques. This RECORD is dedicated to that purpose and should, therefore, be of interest to planners, designers, and construction and soils engineers.

The first 2 papers cover studies of various stabilizing materials and consider their relative merits in improving the natural soil. Using the system or mission approach, Epps, Dunlap, Currin, and Gallaway have developed a classification and indexing to aid in simplifying the selection of the stabilizer to be used; while Stewart, Chu, and Fletcher evaluate compactive field studies of sections stabilized by cement, lime, or phosphoric acid.

The consideration of asphalt-treated materials in the design process for a roadway structure indicates that certain properties of the material should be predictable. Hadley, Hudson, and Kennedy describe a method for predicting the tensile properties (modulus of elasticity, Poisson's ratio, and failure strains) and for estimating their changes with changes in the independent variables that affect them. Dunn and Salem present the effects of items such as emulsion content, viscosity, initial moisture content, and addition of various fillers on the shear strength of a sand stabilized with a cationic bitumen emulsion.

The papers by Kennedy and Moore and by Moore, Kennedy, and Kozuh deal with the related problems of determining the properties of lime-treated materials so that they can be properly considered in the design process and developing methods for estimating these properties from any of several different test methods. Moore and Jones, delving into some of the basic mineralogical relationships, present some very interesting findings, the most important of which concerns the inhibitory effect on stabilization of finely divided iron intimately distributed throughout a soil mass.

Field studies that correlated well with predictive theory for stresses and deflection in cement-stabilized soil pavements are presented by Wang and Mitchell. These will furnish a basis for a pavement thickness design that will limit the critical stresses and strains within the pavement and subgrade to acceptable values. Kennedy, Moore, and Anagnos present correlations that can be used to estimate the indirect tensile strength of cement-treated materials from unconfined compressive strengths or cohesiometer values or both. These tensile properties are to be used in the design of subbases. Cracking in soil-cement bases has been studied by George. Extending theoretical studies to small-scale model experiments, the author indicates the various factors that influence crack intensity and reports on the promising effects of lime or lime-sugar additives to reduce cracking.

CONTENTS

SOIL STABILIZATION: A MISSION ORIENTED APPROACH

J. A. Epps, W. A. Dunlap, and B. M. Gallaway, Civil Engineering Department, Texas A&M University, College Station; and
D. D. Currin, U.S. Department of the Air Force, Kirtland Air Force Base, New Mexico

The widespread use of chemical additives for improving the physical properties of soils and soil-aggregate systems has emphasized the critical need for a classification and indexing system to simplify the selection of the most desirable chemical to be used for the existing environmental conditions and service demands. Such a system is described in this paper. The soil stabilization indexing system is subdivided into parts dealing separately with lime, portland cement, bituminous materials, and combinations of these materials. The different criteria for the use of each of these stabilizers are described in detail with extensive references to the literature. A series of flow charts have been developed that can be used in selecting the type and the amount of stabilizer for a given soil.

•THE U.S. Department of the Air Force demands and utilizes a broad array of airfield pavement types, ranging from very austere temporary runways in forward combat zones to well-engineered, heavy-duty runways designed for the most up-to-date aircraft. Because many of the existing pavements were built in the early 1930's, a continual program of maintenance and reconstruction is carried so that the airfields can accommodate modern aircraft. New construction is also mandatory, and this includes permanent facilities as well as limited-life pavement systems, many of which are constructed within very severe time constraints. Expedient construction must take full advantage of on-site construction materials because all additional materials and equipment must be airlifted in to ensure rapid response.

The attractive engineering and economic benefits of soil stabilization make it necessary that this construction alternative be considered. Yet, in many cases, the engineer has no past experience or specialized training in soil stabilization techniques. To alleviate this problem, an index system is required that will allow the engineer to select the appropriate type and amount of stabilizer. The use of the index system in the field should require determination of relatively simple properties of the soil. These soil properties, together with suitable use factors and environmental data, should be used as input to the index system.

AIR FORCE SOIL STABILIZATION INDEX SYSTEM

An overall systematic approach was used in developing the Air Force soil stabilization index system (SSIS). The development of this system, shown in Figure 1, is discussed in this section (1).

Type of Stabilization

Chemical stabilization is of primary concern in the SSIS. However, both chemical and mechanical stabilization must be considered and the alternatives evaluated.

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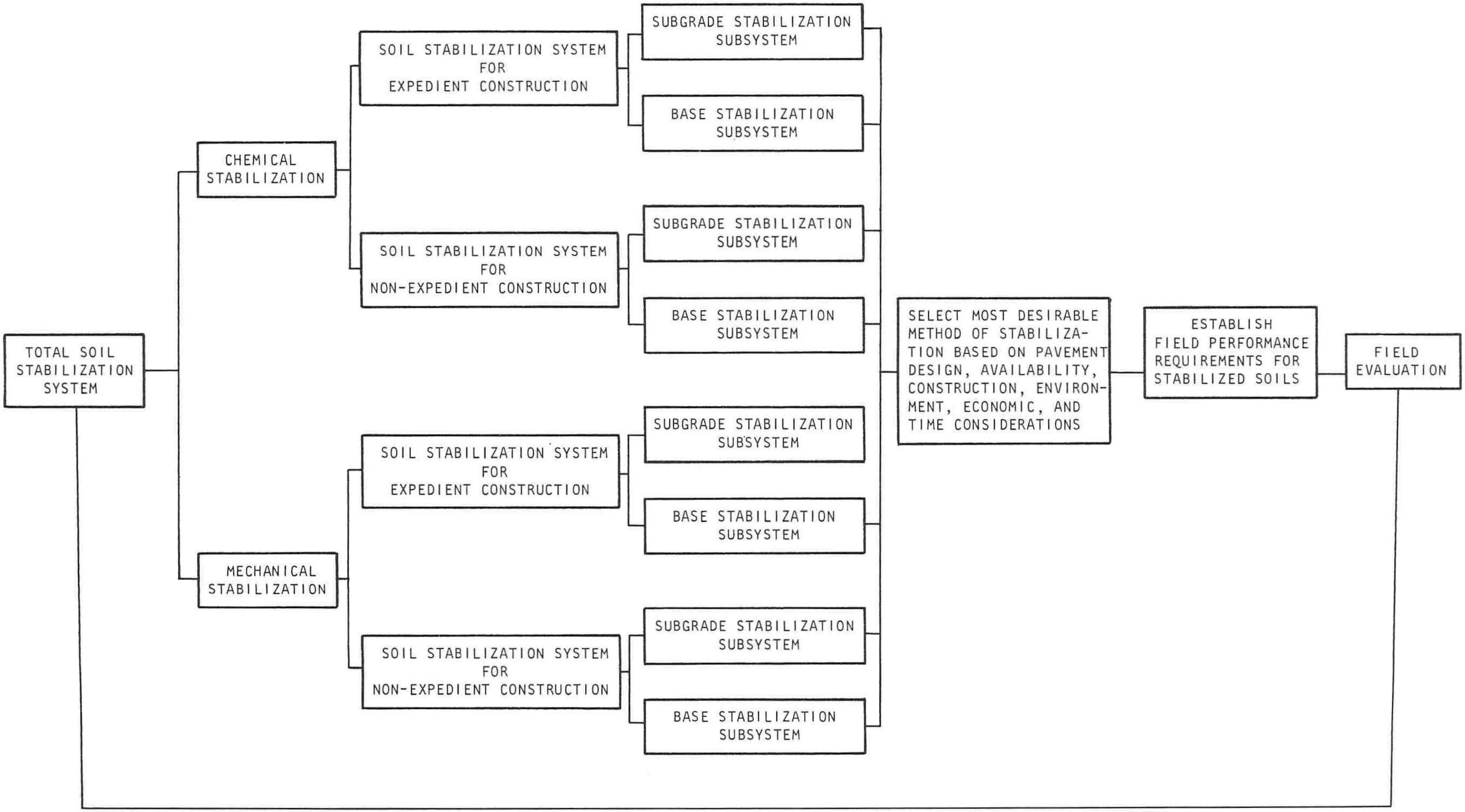


Figure 1. The Air Force soil stabilization index system.

Use Factors

The soil stabilization system should be capable of being utilized for (a) theater of operations use on both expedient and nonexpedient pavements and (b) zone of interior use on permanent pavements.

Expedient refers to short-lived, high-risk, rapidly constructed pavements, whereas nonexpedient and permanent pavements have a longer life and require an extended construction period. The major difference between nonexpedient and permanent pavements is that the latter would probably be constructed by civilian firms and the design lead time would allow more thorough and detailed investigation of stabilization alternatives. Permanent construction is identical to that used by state highway departments for primary roads, and the index system for nonexpedient construction supplies a "jumping-off" point for investigations in permanent construction.

Figure 1 shows another way in which use factors are entered in the index system by specifying different subsystems for subgrade and base course stabilization. Subbases are not considered directly, but they may fall either in the subgrade or base course subsystems depending on the material type and desired strength characteristics.

Environmental Factors

Environmental factors might influence the ultimate durability and suitability of the stabilized soil. They are based primarily on climatological effects. Both rainfall and temperature must be considered because either can significantly influence the type and amount of stabilizer used as well as the time of the year in which certain stabilizers can be used.

Construction Factors

Military engineers faced with hasty construction in the theater of operations usually are faced with limited equipment also. Knowledge of the type of equipment required for a certain stabilization task may prove to be a valuable planning tool not only in anticipating the type of equipment necessary to perform a stabilization task but also in eliminating the use of a particular stabilizer if adequate equipment and time are not available.

Field Performance Requirements for Stabilized Soils

The desired performance of the stabilized soils is established by the Air Force. In most cases, this is based on anticipated life of the structure and allowable time for construction. Examples of this information include the recent mobility concepts and various other operational requirements that have been developed by the Air Force.

Field Evaluation

The verification of the index system for soil stabilization must ultimately come from the user, i.e., the Air Force and its military partners. On pavement projects where stabilization has been used, adequate construction records and follow-up evaluations will be absolutely necessary to verify the adequacy of the stabilized sections. Continual evaluations of stabilized sections that are already in place will also aid in evaluating the ultimate performance of the index system.

Finally, it should be stressed that the SSIS is not a substitute for structural pavement design. In its present form, it will not indicate to an engineer whether a layer should be stabilized or whether there are structural advantages of stabilizing one layer instead of another. Rather, the role of the index system is this: If the engineer decides to use stabilization, then he should be able to use the index system to tell him what kind of stabilization to use and how much stabilizer he should use. Soils that are not amenable to stabilization can be so identified in the index system. If other circumstances, such as climatic conditions or lack of appropriate equipment, rule out the possibility of stabilization, the index system can also provide this information.

GENERAL REQUIREMENTS FOR SELECTING STABILIZERS

Several guides have been published that assist the engineer in the selection of a stabilizer for a particular soil (2, 3). These guides indicate that selection of the stabilizer is dependent on the location of the stabilized layer in the pavement as well as the soil type. Systems have been developed for both base course and subgrade stabilization (1), although only the base course stabilization system will be presented in this paper.

Both the Unified Soil Classification System and the AASHO Soil Classification System have been utilized to select soil stabilizers (4, 5). Because both grain size and Atterberg limits are necessary inputs to classify soils according to either system, these 2 parameters were used for the initial separation of the soils into specific categories. In particular, the percentage passing the No. 200 sieve and the plasticity index (PI) were selected.

Specific guidelines for stabilizer selection are also available from literature published by consumer, producer, user, and general interest groups. These guidelines are discussed here in detail for lime, cement, bituminous materials, and combinations of these stabilizers.

Criteria for Lime Stabilization

Lime will react with most medium, moderately fine, and fine-grained soils to decrease plasticity, increase workability, reduce swell, and increase strength (6). In general terms, the soils that are most reactive to lime include (7) clayey gravels, silty clays, and clays. All soils classified by AASHO as A-5, A-6, and A-7 and some soils classified as A-2-6 and A-2-7 are most readily susceptible to stabilization with lime. Soils classified according to the Unified System as CH, CL, MH, ML, CL-ML, SC, SM, GC, and GM should be considered as potentially capable of being stabilized with lime.

Robnett and Thompson (6), based on experience gained with Illinois soils, have indicated that lime may be an effective stabilizer with clay contents ($< 2\mu$) as low as 7 percent; and, furthermore, soils with a PI as low as 8 can be satisfactorily stabilized with lime in certain instances (8). Armed forces criteria (2) indicate that the PI should be greater than 12, while representatives of the National Lime Association (9) indicate that a PI greater than 10 would be a reasonable lower limit to utilize.

In view of these suggested criteria, it is believed that the PI of the soil should have a lower limit of 10 to ensure that a reasonable degree of certainty will exist for successful stabilization with lime.

Criteria for Cement Stabilization

The Portland Cement Association (10, 11) indicates that all types of soils can be stabilized with cement. However, well-graded granular materials that possess a floating aggregate matrix have given the best results (12).

Limits on PI have been established by the armed forces (2), depending on the soil type. The PI should be less than 30 for sandy and gravelly materials and less than 20 for the fine-grained soils. These limitations are necessary to ensure proper mixing of the stabilizer in the soil.

Information developed by the Federal Highway Administration (5) indicates that cement should be used as a stabilizer for materials with less than 35 percent passing the No. 200 sieve and with a PI less than 20. Thus, this implies that A-2 and A-3 soils can be best stabilized by cement, while A-5, A-6, and A-7 soils can be best stabilized by lime.

The authors have selected a maximum PI of 30 for those soils to be stabilized with cement.

Criteria for Bituminous Stabilization

The majority of soil-bituminous stabilization has been performed with asphalt cement, cutback asphalts, and emulsified asphalts. For this reason, only these types of bituminous stabilizers are considered.

Some of the earliest criteria for bituminous stabilization were developed by the HRB Committee on Soil-Bituminous Roads. These criteria were revised and published by Winterkorn (13). Other criteria have been presented by the American Road Builders Association (14), The Asphalt Institute (15, 16), Herrin (17), Chevron Asphalt Company (18), Douglas Oil Company (19), and the U.S. Department of the Navy (20). Although these criteria were developed for particular types of bituminous stabilizers (i.e., soil-bitumen made with cutback asphalt), they are given in a single table (Table 1) for comparison purposes.

Current trends indicate that stabilization with asphalt cements is gaining widespread application. Requirements for aggregate grading and mixture properties of mixes containing asphalt cement have recently been summarized by the HRB Committee on Bituminous Aggregate Bases (21). This survey of criteria together with data published by the armed forces (22) suggests that soils that are nearly nonplastic and contain less than 18 percent passing the No. 200 sieve are most suitable for hot-mix asphalt cement stabilization.

Based on these criteria, a limit of 20 percent passing the No. 200 sieve, a PI less than 6, and the product of PI and the minus No. 200 material less than 60 have been utilized for selecting soils suitable for stabilization by asphalt. Less stringent requirements have been used in conjunction with the other stabilization subsystems developed for the Air Force (1).

Criteria for Combination Stabilizers

Combination stabilization is here defined specifically as lime-cement, lime-asphalt, and lime-fly ash. Because lime-fly ash stabilization is not expected to be a common stabilization method used by the Air Force, it will not be incorporated into the index system. The purpose of using combination stabilizers (lime and then one or the other stabilizers) is to reduce plasticity and increase workability so that the soil may be effectively stabilized by the second agent or additive.

Robnett and Thompson (23) have reviewed the literature and have suggested that soils that may be treated by these combination stabilizers are those classified by AASHO as A-6 and A-7 and certain soils classified as A-4 and A-5.

Based on these findings, it has been suggested that these combination stabilizers be utilized with materials that have greater than 35 percent passing the No. 200 sieve and that quantities of lime be used sufficient in magnitude to ensure that the PI is less than the established criteria for either cement or asphalt stabilization as appropriate.

These criteria together with appropriate environmental and construction precautions as given in Table 2 have been used to establish the base course stabilization system shown in Figure 2.

This stabilization system separates soils into various groups so the engineer may select the stabilizer suitable for use within these particular groups. This system will not, however, indicate the amount of stabilizer that must be used for a particular soil. The following discussion will suggest criteria that will allow the development of appropriate subsystems for the determination of stabilizer quantities.

TABLE 1

CRITERIA DEVELOPED FOR BITUMINOUS STABILIZATION

Developer	Percent Passing No. 200 Sieve	Plasticity Index	Plasticity Index × Percent Passing No. 200 Sieve
Winterkorn	8 to 50	18	
American Road Builders Association	0 to 35	10	
Herrin	0 to 30	10	
The Asphalt Institute, Pacific Coast Division	3 to 15	6	60
Chevron Asphalt Company	0 to 25	Nonplastic	72
Douglas Oil Company	0 to 30	7	

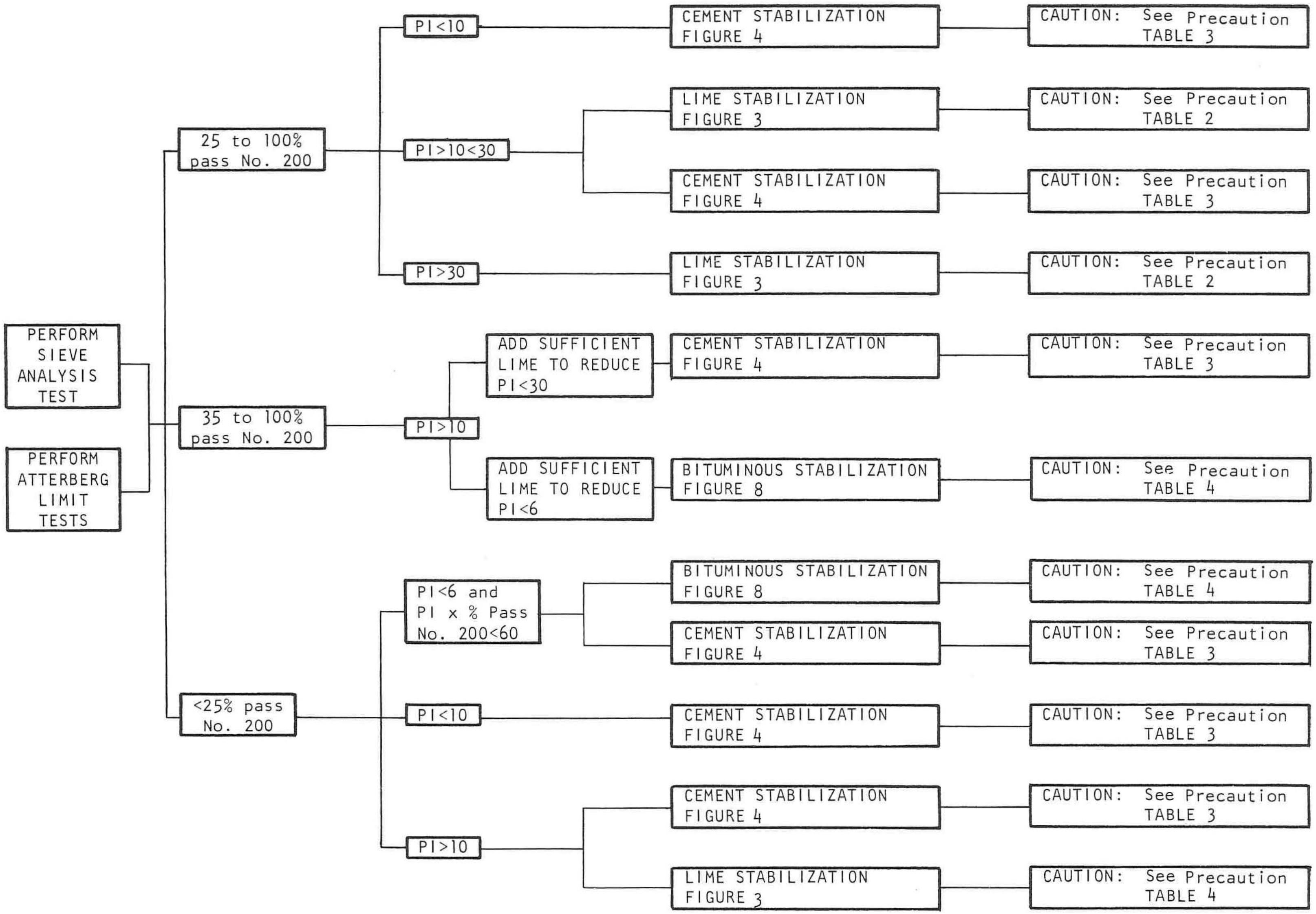


Figure 2. Selection of stabilizer for nonexpedient base construction.

TABLE 2

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

Stabilization	Environmental or Construction	Precaution
Lime	Environmental	If the soil temperature is less than 40 F and is not expected to increase for 1 month, chemical reactions will not occur rapidly and, thus, the strength gain of the lime-soil mixture will be minimal.
		Lime-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist any expected freeze-thaw cycles.
	Construction	Heavy vehicles should not be allowed on the lime-stabilized soil for 10 to 14 days after construction.
Cement	Environmental	If the soil temperature is less than 40 F and is not expected to increase for 1 month, chemical reactions will not occur rapidly and, thus, the strength gain of the cement-soil mixture will be minimal.
		Cement-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist the expected freeze-thaw cycles.
		Construction during periods of heavy rainfall should be avoided.
	Construction	Heavy vehicles should not be allowed on the cement-stabilized soil for 7 to 10 days after construction.
Bituminous	Environmental	When asphalt cements are used, construction should be allowed only when proper compaction is possible. If thin lifts are being placed, the air temperature should be 40 F and rising. Adequate compaction can be obtained at freezing temperatures.
		When cutbacks and emulsions are being used, the air temperature and soil temperature should be above freezing.
		Bituminous materials should completely coat the soil particles prior to compaction.
	Construction	Central batch plants, together with other specialized equipment, are necessary for bituminous stabilization with asphalt cements.
		Hot, dry weather is preferred for all types of bituminous stabilization.

DESIGN SUBSYSTEMS

Numerous research publications and technical guides are available to aid the engineer in the selection of criteria to determine the amount of stabilizer. A wide variety of test methods have been proposed; however, quantitative criteria are not always available. The criteria discussed here are for establishing the design subsystems aimed at determining appropriate stabilizer quantities for lime, cement, and bituminous stabilization.

Lime Stabilization

Selection of Appropriate Soils—The preceding section discussed the general requirements of the soil with respect to gradation and plasticity. However, there are other physicochemical features that must be considered in determining whether lime will react with a soil.

Thompson (24) has defined soils as being lime-reactive if they display significant strength increase (measured by unconfined compressive strength) when treated with lime. Soils that are not lime-reactive according to this definition are not necessarily unimproved by the addition of lime because lime may still decrease the plasticity, decrease the susceptibility to water, and enhance the overall engineering behavior of the soil. However, because improved load-bearing characteristics are desired in the stabilization index system, strength will be a major consideration here.

Factors that may prohibit soils from being lime-reactive include soil pH and the presence of organics and sulfates. Soils with a pH less than 7 may not be lime-reactive, although some soils with pH values as low as 5.7 have reportedly been effectively stabilized with lime (24). It has also been reported that soils with organic carbon contents exceeding about 1 percent are not satisfactorily lime-reactive (24). In addition, experience has shown that the presence of significant amounts of sulfates diminishes the effectiveness of lime.

It has been reported that A-horizon soils in Illinois do not satisfactorily react with lime (24), and similar reports have been made on other soils. This is probably the result of high organic contents in the upper horizon. Poorly drained soils often are the most reactive to lime, possibly because of the higher pH and the availability of lime-reactive constituents, such as unweathered soil minerals.

Selection of Type of Lime—Lime is generally used as an all-encompassing term to denote either slaked (hydrated) lime or quicklime. Both calcitic lime and dolomitic (high magnesium) lime are available in the United States. Although there is some disagreement as to whether the type of lime influences the strength of lime-soil mixtures (25), the selection of the lime type is usually predicated by availability of the stabilizer and safety requirements of the particular job.

Selection of Lime Quantity—There are fewer definitive criteria for evaluating the correct quantity of lime than for cement or bituminous materials.

Eades and Grim (26) have proposed a short-cut test where the appropriate lime content is that which will produce a minimum pH of 12.4 one hour after mixing. This test has not been validated for soils on a worldwide basis and should be used with caution.

Most authors have reported that a minimum of 3 percent lime is necessary to produce adequate reactions in the field (27). The Air Force (28) suggests that 2, 3, and 5 percent lime be used in coarse soils (those containing 50 percent or less passing the No. 200 sieve) while 3, 5, and 7 percent be tried for fine-grained soils (greater than 50 percent passing the No. 200 sieve). The National Lime Association recommends 3, 5, and 7 percent lime in trial mixtures (27). With the exception of the pH test described, the lime content must generally be determined by trial mixtures with the amount of lime being the minimum required to produce the desired reactions.

Methods of Evaluating Soil-Lime Mixtures—Several types of tests have been proposed for evaluating soil-lime mixtures. These include, but are not limited to, unconfined compressive strength, California bearing ratio, flexural fatigue strength, triaxial compressive strength, tests yielding elastic properties, cohesiometer values, and freeze-thaw and wet-dry tests. Most of these tests are not used routinely, and satisfactory criteria are not generally available. Some of the most reliable data are based on unconfined compressive strengths developed from research done by Thompson (29). Table 3 gives his results.

Durability, the ability of a material to retain stability and integrity over years of exposure to service and weathering, is perhaps the most difficult to determine. Of the many tests developed, only a modified freeze-thaw test shows substantial merit (30).

Figure 3, the lime stabilization subsystem, has been developed from these criteria.

TABLE 3

TENTATIVE LIME-SOIL MIXTURE COMPRESSIVE STRENGTH REQUIREMENTS

Anticipated Use	Residual Strength Requirement[b] (psi)	Strength Requirements for Various Anticipated Service Conditions[a]			
		8-Day Extended Soaking (psi)	Cyclic Freeze-Thaw[c] (psi)		
			3 Cycles	7 Cycles	10 Cycles
Modified subgrade	20	50	50	90 50[d]	120
Subbase					
Rigid pavement	20	50	50	90 50[d]	120
Flexible pavement					
10-in. cover[e]	30	60	60	100 60[d]	130
8-in. cover[e]	40	70	70	110 75[d]	140
5-in. cover[e]	60	90	90	130 100[d]	160
Base	100[f]	130	130	170 150[d]	200

[a]Strength required at termination of filed curing (following construction) to provide adequate residual strength.
[b]Minimum anticipated strength following first winter exposure.
[c]Number of freeze-thaw cycles expected in the lime-soil layer during the first winter of service.
[d]Freeze-thaw strength losses are based on 10 psi/cycle except for these 7-cycle values that are based on a previously established regression equation.
[e]Total pavement thickness overlying the subbase; requirements are based on Boussinesq stress distribution; rigid pavement requirements apply if cemented materials are used as base courses.
[f]Flexural strength should be considered in thickness design.

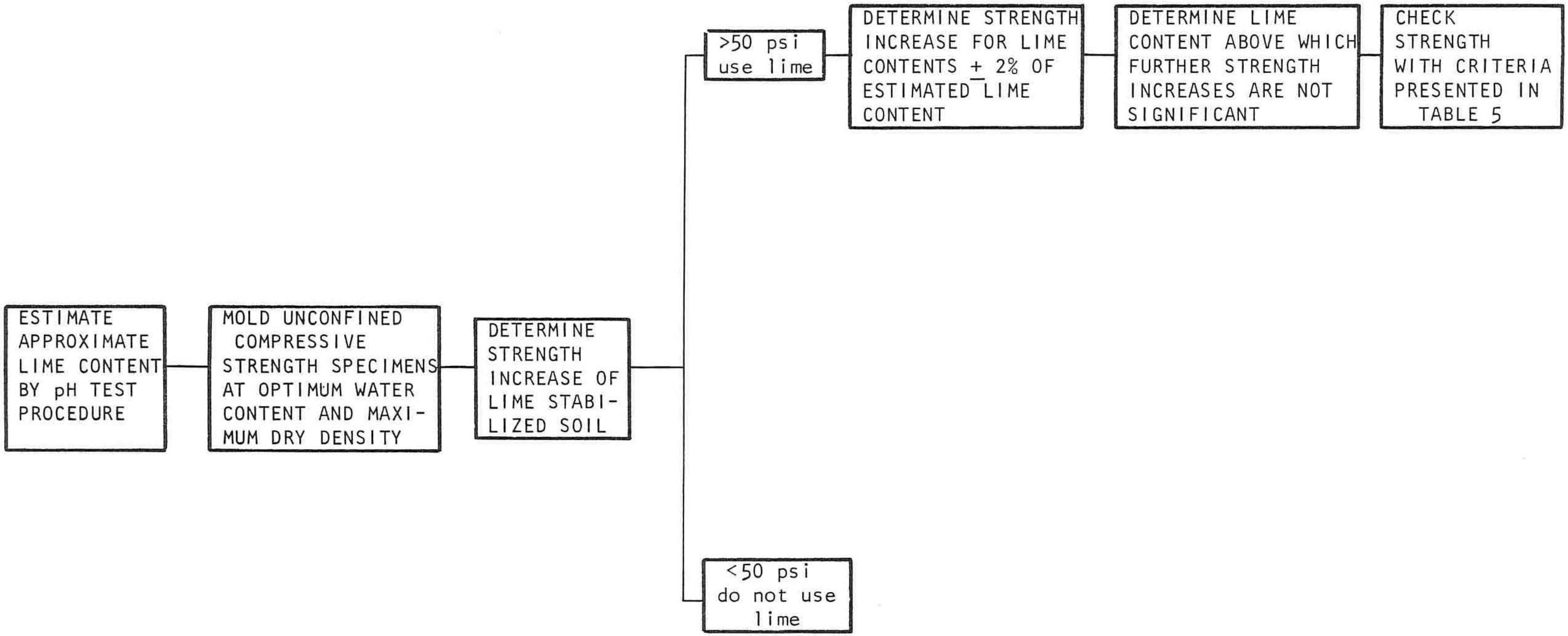


Figure 3. Subsystem for base course stabilization with lime.

Cement Stabilization

Information as to general requirements such as gradation and Atterberg limits have been discussed previously. Most research and construction with cement-soil mixtures has been performed on soils that have been classified according to the AASHO classification system. Experience has shown that this approach is satisfactory; but, it does not include important soil properties such as clay type, soil pH, organic content, and soil sulfate content that may influence the suitability of a soil for cement stabilization. These effects are discussed in this section.

Effects of pH, Organics, and Sulfates—The Road Research Laboratory has shown a general trend of strength increase with soils possessing high pH values. For pH values greater than 7, no ill effects on strength were noted (31). The Portland Cement Association has conducted pH tests on soils, but it has found no general correlation between pH and performance (32).

Two tests have been proposed to assess the effects of organics on soil-cement strength. The Portland Cement Association (33) has suggested the use of the calcium adsorption test to determine the presence of organics in sandy soils, but this test should not be used for clay soils. Additional research conducted by the Portland Cement Association (32, 33) has shown that the standard colorimetric tests will not identify the presence of organics satisfactorily.

A satisfactory method for determining the presence of active organic matter, according to MacLean and Sherwood (31), is the pH test conducted on a soil-cement paste 15 minutes after mixing. This test essentially indicates the reactivity of the soil with cement; however, the reactivity is not solely a function of the organic content (32, 34), but it is dependent on both the organic content and the type of organics (35).

Studies conducted by Sherwood (36) have indicated that sulfate contents in soils in excess of 0.5 to 1.0 percent reduce the strength of soil-cement mixtures. Similarly, sulfate concentrations in water in excess of 0.05 percent create strength loss. For these reasons the sulfate content of the soil should be ascertained.

Type of Cement—The influence of the type of cement on the properties of soil-cement mixtures has been examined by several investigators (36, 37, 38, 39). In general, Types I, II, III, and V produce only small differences in behavior for most soils. Thus, because of its general availability and economy, it is recommended that Type I cement be utilized.

Selection of the Cement Quantity—Research performed by the Portland Cement Association (10, 40, 41, 42) on more than 2,000 soils provides data for determining cement contents for various types of soils. Cement contents for subsurface soils are given in Table 4 (10). Requirements for soils in various horizons are also specified by the Portland Cement Association.

TABLE 4

CEMENT REQUIREMENTS FOR VARIOUS SOILS

AASHO Soil Classification	Unified Soil Classification[a]	Usual Range in Cement Requirement[b] Percent by Volume	Usual Range in Cement Requirement[b] Percent by Weight	Estimated Cement Content Used in Moisture-Density Test (percent by weight)	Cement Content for Wet-Dry and Freeze-Thaw Tests (percent by weight)
A-1-a	GW, GP, GM, SW, SP, SM	5 to 7	3 to 5	5	3 to 5 to 7
A-1-b	GM, GP, SM, SP	7 to 9	5 to 8	6	4 to 6 to 8
A-2	GM, GC, SM, SC	7 to 10	5 to 9	7	5 to 7 to 9
A-3	SP	8 to 12	7 to 11	9	7 to 9 to 11
A-4	CL, ML	8 to 12	7 to 12	10	8 to 10 to 12
A-5	ML, MH, OH	8 to 12	8 to 13	10	8 to 10 to 12
A-6	CL, CH	10 to 14	9 to 15	12	10 to 12 to 14
A-7	OH, MH, CH	10 to 14	10 to 16	13	11 to 13 to 15

[a]Based on U.S Air Force recommendations (2).

[b]For most A-horizon soils, the cement content should be increased 4 percentage points if the soil is dark gray to gray and 6 percentage points if the soil is black.

Methods of Evaluating Soil-Cement Mixtures—Various types of tests have been used to evaluate the properties of soil-cement mixtures (43). These methods include unconfined compressive strength, flexural strength, modulus of elasticity, California bearing ratio, plate bearing value, fatigue, R-value, and freeze-thaw and wet-dry tests.

Many of these test methods have not been used extensively, and satisfactory criteria are not available. However, the Portland Cement Association recommends the use of freeze-thaw and wet-dry tests and has established criteria (Table 5) for these tests.

The design subsystem for cement based on these criteria is shown in Figure 4.

TABLE 5

PORTLAND CEMENT ASSOCIATION CRITERIA FOR SOIL-CEMENT MIXTURES USED IN BASE COURSES

AASHO Soil Classification	Unified Soil Classification[a]	Soil-Cement Weight Loss During 12 Cycles of Either Wet-Dry or Freeze-Thaw Test (percent)
A-1	GW, GP, GM, SW, SP, SM	≤14
A-2-4, A-2-5	GM, GC, SM, SC	
A-3	SP	
A-2-6, A-2-7	GM, GC, SM, SC	≤10
A-4	CL, ML	
A-5	ML, MH, OH	
A-6	CL, CH	≤7
A-7	OH, MH, CH	

[a]Based on correlation presented by U.S. Air Force (2).

Bituminous Stabilization

A bituminous binder in 1 of 3 forms is generally used; the forms include cutbacks, emulsions, or cements. An indication of the type of bitumen to use for certain types of soils has been suggested by The Asphalt Institute (15), Herrin (17), the U.S. Navy (20), the Air Force (28), and Chevron Asphalt Company (18). Selection of the proper bituminous stabilizer should depend on the grain-size distribution in addition to the function of the stabilized layer in the pavement system. Table 6, adapted from Herrin and prepared by using the soil gradings also suggested by Herrin (17), and Table 7 give data regarding bitumen stabilization.

Asphalt Cement—Criteria used for selection of the binder viscosity and the quantity of cement for base stabilization vary among state highway departments (21), and a suitable method based on highway experience is not available. The armed forces, however, base the selection of asphalt cement viscosity or grade on the pavement temperature index. Their recommendations have been altered and are used in the design subsystem (Table 8).

The quantity of asphalt can be estimated on a surface area and particle surface characteristic concept such as the California CKE method, or the quantity can be estimated from experience. Data given in Table 9 can be used to obtain a preliminary estimate of asphalt content, but these quantities are a guide only. Final selection should be based on a test performed on the asphalt-aggregate mixture.

A recent summary of state practices (21) indicates that both Hveem and Marshall tests are popular evaluation methods among state highway departments and that criteria

TABLE 6

SELECTION OF A SUITABLE TYPE OF BITUMEN FOR SOIL STABILIZATION PURPOSES

Mix	Sand-Bitumen	Soil-Bitumen	Crushed Stones and Sand-Gravel-Bitumen
Hot	Asphalt cements 60 to 70 hot climate 85 to 100 120 to 150 cold climate		Asphalt cements 45 to 50 hot climate 60 to 70 85 to 100 cold climate
Cold	Cutbacks See Figure 5	Cutbacks See Figure 5	Cutbacks See Figure 5
Emulsions	Emulsions See Table 11 See Figures 6 and 7 to determine whether cationic or anionic emulsion should be used	Emulsions See Table 11 See Figures 6 and 7 to determine whether cationic or anionic emulsion should be used	Emulsions See Table 11 See Figures 6 and 7 to determine whether cationic or anionic emulsion should be used

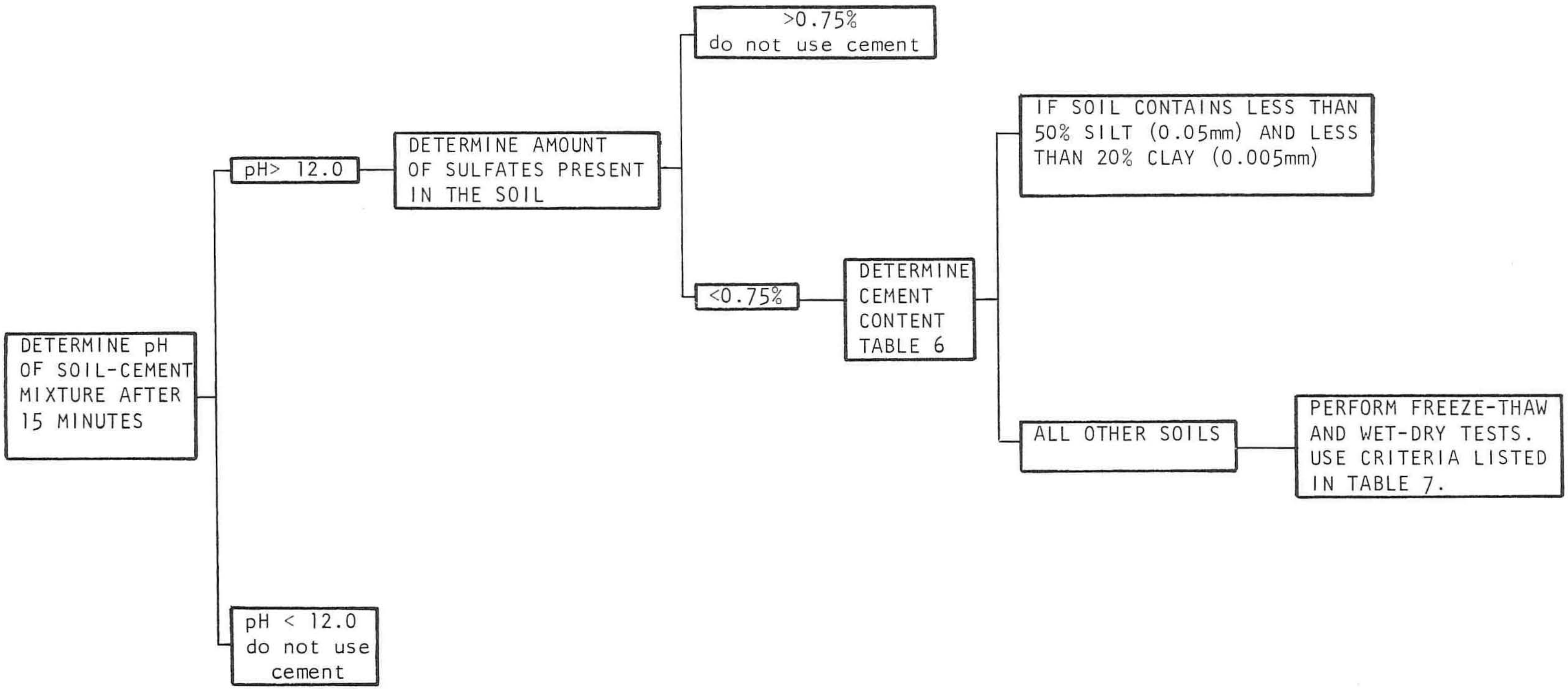


Figure 4. Subsystem for base course stabilization with cement.

TABLE 7

ENGINEERING PROPERTIES OF MATERIALS SUITABLE FOR BITUMINOUS STABILIZATION

Item	Sand-Bitumen	Soil-Bitumen		Sand-Gravel-Bitumen
Gradation (percent passing)				
1½-in. sieve				100
1-in. sieve	100			
¾-in. sieve				60 to 100
No. 4 sieve	50 to 100		50 to 100	35 to 100
No. 10 sieve	40 to 100			
No. 40 sieve			35 to 100	13 to 50
No. 100 sieve				8 to 35
No. 200 sieve	5 to 12	Good	3 to 20	
		Fair	0 to 3 and 20 to 30	0 to 12
		Poor	>30	
Liquid limit		Good	<20	
		Fair	20 to 30	
		Poor	30 to 40	
		Unusable	>40	
Plasticity index	<10	Good	<5	
		Fair	5 to 9	
		Poor	9 to 15	<10
		Unusable	>12 to 15	

Note: Includes slight modifications later made by Herrin (17).

vary from state to state. Marshall method criteria utilized by the armed forces (2) are given in Table 10 (2). The criteria listed for asphaltic-concrete binder course are indicated for use with coarse-graded, hot-mix base courses, while separate criteria are given for sand-asphalt. The Air Force has also indicated that the asphalt content determined by the Marshall method should be altered depending on the pavement temperature index. However, this criterion, which was developed for surface courses, does not appear to be warranted for base courses.

The Asphalt Institute (44) recommends Marshall, Hveem, and Hubbard-Field criteria for use in hot-mix base course design. Specifically, The Asphalt Institute recommends the same criteria that are utilized for surface courses, but with a test temperature of 100 F rather than 140 F. This recommendation applies to regions having climatic conditions similar to those prevailing throughout most of the United States and to bases that are 4 in. or more below the surface.

Zoeph (45) recommends Marshall criteria based on studies conducted in Germany, while McDowell and Smith (46) have recently presented a design procedure based on unconfined compressive strength and air void criteria.

Recently, attempts have been made to develop a more rational approach to pavement design. Among others, Monismith (47) has indicated that elastic and fatigue properties of asphalt-treated base courses should be considered in pavement design. These more rational methods should allow engineers to better assess the engineering behavior of these stabilized materials.

TABLE 8

DETERMINATION OF ASPHALT GRADE FOR BASE COURSE STABILIZATION

Pavement Temperature Index[a]	Asphalt Grade (penetration)
Negative	100 to 120
0 to 40	85 to 100
40 to 100	60 to 70
100 or more	40 to 50

[a]The sum, for a 1-year period, of the increments above 75 F of monthly averages of the daily maximum temperatures. Average daily maximum temperatures for the period of record should be used where 10 or more years of record are available. For records of less than 10-year duration, the record for the hottest year should be used. A negative index results when no monthly average exceeds 75 F. Negative indexes are evaluated merely by subtracting the largest monthly average from 75 F.

TABLE 9

SELECTION OF PRELIMINARY ASPHALT CEMENT CONTENT FOR BASE COURSE CONSTRUCTION

Aggregate Shape and Surface Texture	Asphalt by Weight of Dry Aggregate (percent)
Rounded and smooth	4
Angular and rough	6
Intermediate	5

TABLE 10

CRITERIA OF MARSHALL METHOD FOR DETERMINATION OF OPTIMUM BITUMEN CONTENT

Test Property	Type of Mix	Point on Curve		Criteria	
		For 100-psi Tires[a]	For 200-psi Tires[a]	For 100-psi Tires[a]	For 200-psi Tires[a]
Stability	Asphaltic-concrete surface course	Peak of curve	Peak of curve	500 lb or higher	1,800 lb or higher
	Asphaltic-concrete binder course	Peak of curve[b]	Peak of curve[b]	500 lb or higher	1,800 lb or higher
	Sand asphalt	Peak of curve		500 lb or higher	
Unit weight	Asphaltic-concrete surface course	Peak of curve	Peak of curve	Not used	Not used
	Asphaltic course binder course	Not used	Not used	Not used	Not used
	Sand asphalt	Peak of curve		Not used	Not used
Flow	Asphaltic-concrete surface course	Not used	Not used	20 or less	16 or less
	Asphaltic course binder course	Not used	Not used	20 or less	16 or less
	Sand asphalt	Not used	Not used	20 or less	16 or less
Percentage voids in total mix	Asphaltic-concrete surface course	4 (3)	4 (3)	3 to 5 (2 to 4)	3 to 5 (2 to 4)
	Asphaltic-concrete binder course	5 (4)	6 (5)	4 to 6 (3 to 5)	5 to 7 (4 to 6)
	Sand asphalt	6 (5)	— (-)	5 to 7 (4 to 6)	— (---)
Percentage voids filled with bitumen	Asphaltic-concrete surface course	80 (85)	75 (80)	75 to 85 (80 to 90)	70 to 80 (75 to 85)
	Asphaltic-concrete binder course	70 (75)	60 (65)[b]	65 to 75 (70 to 80)	70 to 80 (55 to 75)
	Sand asphalt	70 (75)	— (--)	65 to 75 (70 to 80)	— (--)

[a]Figures in parentheses are for use with bulk-impregnated specific gravitv (water absorption greater than 2.5 percent).

[b]If the inclusion of asphalt contents of these points in the average causes the voids to fall outside the limits, then the optimum asphalt content should be adjusted so that the voids in the total mix are within the limits.

Criteria currently used by the armed forces for binder course utilizing Marshall mix design methods have been suggested for use.

Cutback Asphalts—The U.S. Navy (20) has suggested that the grade of cutback can be selected based on the percentage of the soil passing the No. 200 sieve and the ambient temperature of the soil (Fig. 5). The Air Force (28) and The Asphalt Institute (15) recommendations are rather general in nature.

Several methods are available to the engineer for selecting the quantity of cutback asphalts. The California CKE method could be utilized as could equations developed in Oklahoma (48) and by The Asphalt Institute (15) based on the surface-area concept. The equation recommended by The Asphalt Institute (15) is

$$p = 0.02(a) + 0.07(b) + 0.15(c) + 0.20(d) \tag{1}$$

where

p = percentage of asphalt material by weight of dry aggregate;
a = percentage of mineral aggregate retained on No. 50 sieve;
b = percentage of mineral aggregate passing No. 50 and retained on No. 100 sieve;
c = percentage of mineral aggregate passing No. 100 and retained on No. 200 sieve; and
d = percentage of mineral aggregate passing No. 200 sieve.

Numerous laboratory tests have been used to determine asphalt contents for cutback and emulsified asphalts. These methods include Hubbard-Field, Hveem stability, Marshall stability, Florida bearing, Iowa bearing, extrusion, unconfined compression, triaxial compression, R-value, and elastic modulus. Mixing methods, curing conditions,

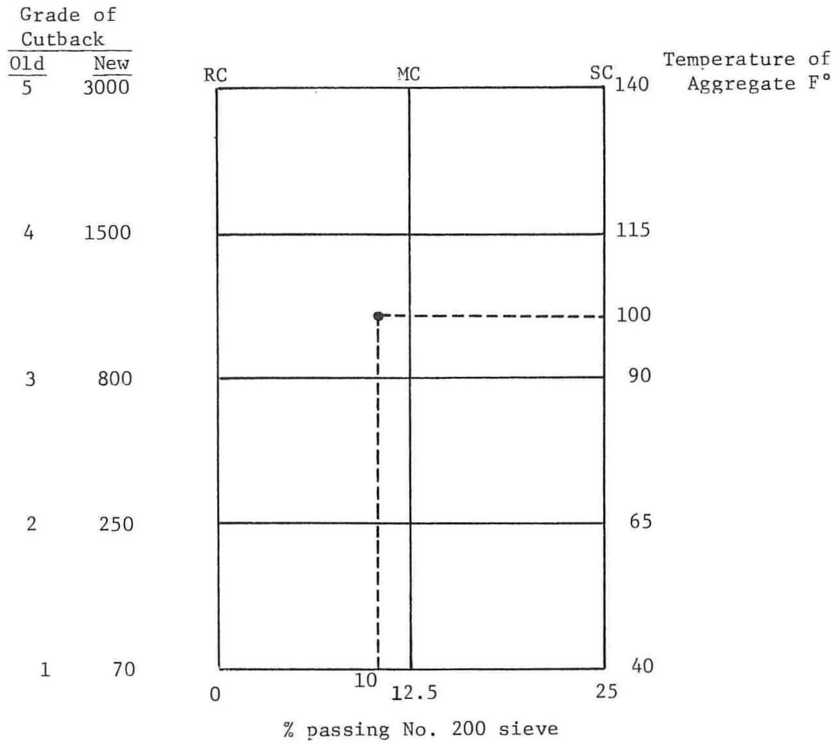
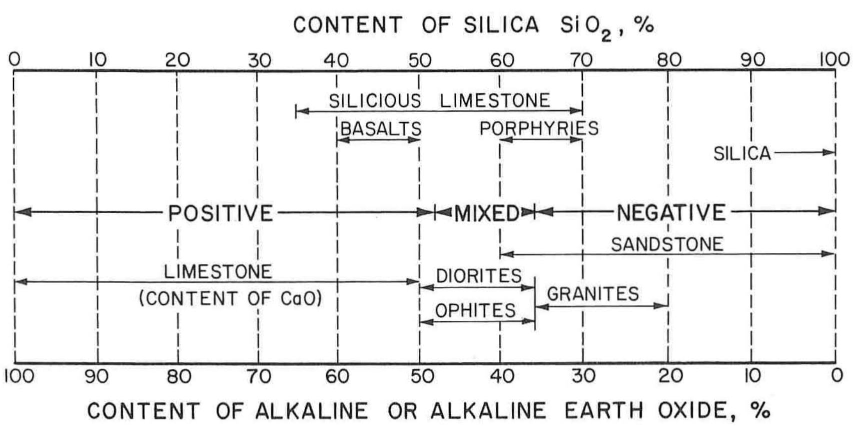


Figure 5. Selection of type of cutback asphalt for stabilization.

rate of loading, and temperature are important variables that must be carefully controlled when these tests are performed.

The Air Force is currently utilizing the extrusion test (28) for mixture design. The unconfined compression test is easy to perform, but sufficient experience to determine adequate criteria for its use is not available.



After Mertens and Wright (52)

Figure 6. Classification of aggregates.

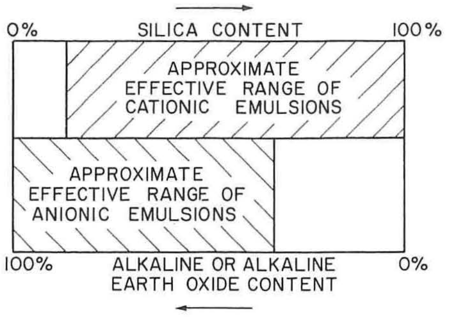
TABLE 11

SELECTION OF TYPE OF EMULSIFIED ASPHALT FOR STABILIZATION

Percent Passing No. 200 Sieve	Relative Water Content of Soil: Wet (5 percent or more)	Relative Water Content of Soil: Dry (0 to 5 percent)
0 to 5	SS-1h (or SS-Kh)	SM-K (or SS-1h)[a]
5 to 15	SS-1, SS-1h (or SS-K, SS-Kh)	SM-K (or SS-1h, SS-1)[a]
15 to 25	SS-1 (or SS-K)	SM-K

Note: Determine from Figures 6 and 7 whether an anionic or a cationic emulsion is to be used.

[a]Soil should be prewetted with water before using these types of emulsified asphalts.



After Mertens and Wright (52)

Figure 7. Approximate effective range of cationic and anionic emulsions on various types of aggregates.

It is important to note that not only are strength or stability criteria necessary for the determination of asphalt content but also a durability criterion is recommended by most agencies. Typical examples of durability tests are the immersion-compression test utilized by Winterkorn (13) and by Riley and Blumquist (49) and moisture vapor susceptibility that is utilized by Chevron Asphalt Company (18), The Asphalt Institute (50), and Finn et al. (51).

Emulsified Asphalts—The selection of the grade of emulsion can be conveniently determined from data given in Table 11, prepared by the U. S. Navy (20). Criteria are based on the percentage passing the No. 200 sieve and the relative water content. The selection of either a cationic or an anionic emulsion should be based on the type of aggregate that is used. Mertens and Wright (52) have developed a method by which aggregate can be classified (Fig. 6) to indicate its probable surface charge and the type of emulsion (anionic or cationic) selected to satisfy particular aggregate surface characteristics (Fig. 7).

A preliminary selection of the quantity of emulsion can be obtained from data given in Table 12 (20). Other methods based on surface area concepts have been used by The Asphalt Institute (15) and Bird (53). The final selection of the quantity should be based on laboratory testing of the asphalt-soil mixture. Because the armed forces are equipped to perform Marshall tests, and apparently a better testing method with proven field performance is not available, the Marshall method with criteria suggested by

TABLE 12

EMULSIFIED ASPHALT REQUIREMENT

Percent Passing No. 200 Sieve	Pounds of Emulsified Asphalt per 100 lb or Dry Aggregate When Percentage Passing No. 10 Sieve Is: 50 or Less	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

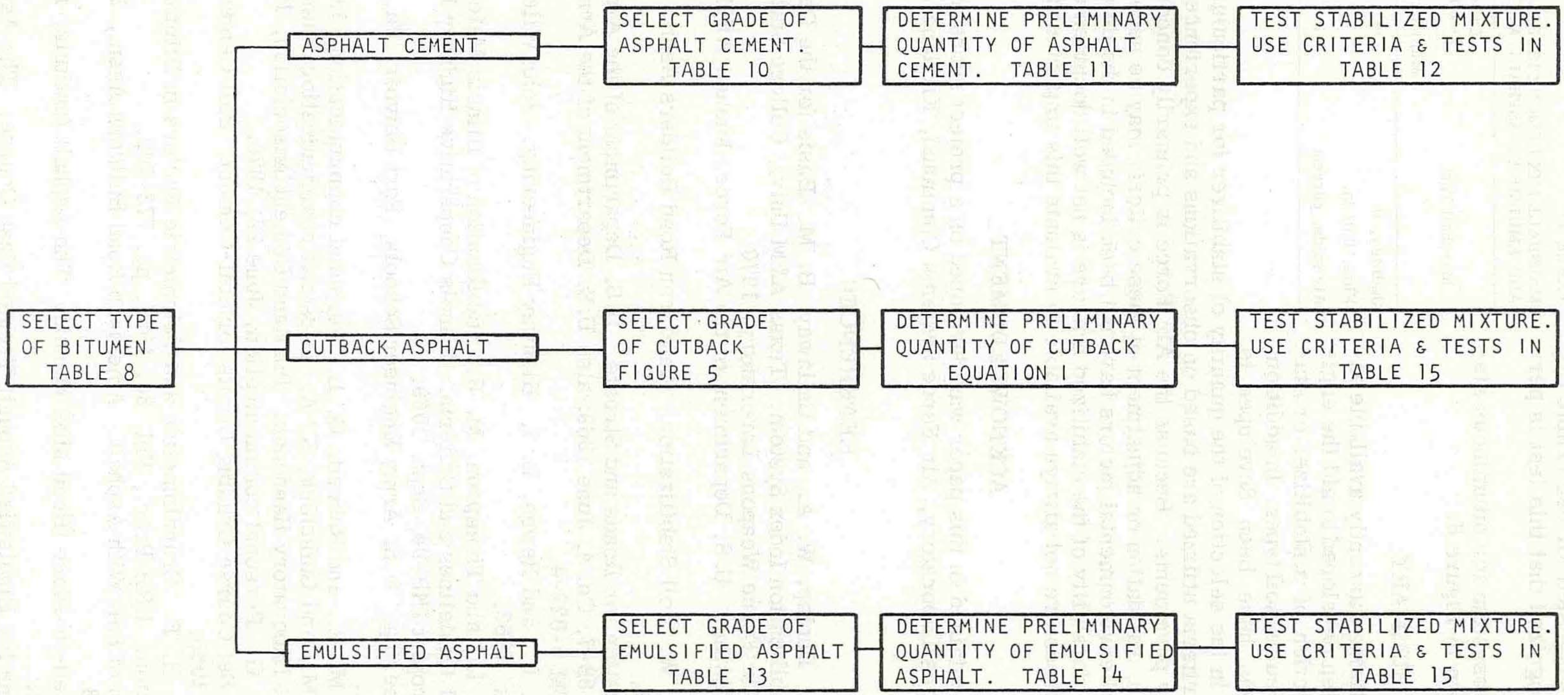


Figure 8. Subsystem for base course stabilization with bituminous materials.

Lefebvre (54) is suggested for use (Table 13). It should be recognized that this test is performed at 77 F.

The design subsystem for bituminous stabilization is shown in Figure 8.

TABLE 13

MARSHALL MIX DESIGN CRITERIA FOR CUTBACK AND EMULSIFIED ASPHALT MIXTURES

Marshall Test	Criteria for a Test Temperature of 77 F	
	Minimum	Maximum
Stability, lb	750	—
Flow, 0.01 in.	7	16
Air voids, percent	3	5

SUMMARY

A system utilizing currently available information has been developed to aid the engineer in the selection of a stabilizer or stabilizers for particular soil types. In addition, design subsystems have been developed to aid the engineer in the selection of the quantity of stabilizer for particular applications.

Many of the criteria utilized are based on observations and experience gained in constructing highway pavements. Because the Air Force is primarily concerned with airfield construction, validation or adjustment of these criteria may be necessary.

Equipment and environmental factors have not been included in the detail desired. In particular, field durability of the stabilized mixture is not well documented and, thus, suitable test methods are not always available to evaluate this important factor.

ACKNOWLEDGMENT

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STABILIZATION OF PIEDMONT SOILS FOR USE AS BASE MATERIAL ON SECONDARY ROAD PROJECTS

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This study was conducted for the purpose of evaluating the effectiveness of various types of stabilizing agents for the micaceous soils found in the Piedmont region. To this end, laboratory investigations with portland cement, hydrated lime, and phosphoric acid as stabilizing agents were carried out, and an experimental road was constructed. Results from these investigations indicate that portland cement is the most effective stabilizing agent and hydrated lime is rated second. Furthermore, the cement requirements of the Piedmont soils appear to be relatively low in comparison with the average values for all types of soil from a wide variety of geologic origin. A simple method for estimating the cement requirement of a given soil was developed. In addition, efforts were made to establish a minimum compressive strength requirement as the criterion for determining the actual cement requirements of the Piedmont soils. In this respect, a tentative criterion of specifying 300 psi as the minimum 7-day compressive strength appears to be satisfactory in designing soil-cement bases for secondary roads in South Carolina. The experimental road was constructed by the type of equipment and procedure that can be expected on secondary road projects. An analysis of test data was made to relate the method of construction with the uniformity and degree of mixing of the stabilized soils.

•PORTLAND cement and hydrated lime have been used for many years in the stabilization of soils for highway construction. While the application of these stabilizing agents for various types of soil has been studied and reported by many investigators, there are relatively few publications presenting specific information in regard to the stabilization of micaceous soils as found in the Piedmont region. A study was recently conducted in South Carolina to evaluate the effectiveness of various stabilizing agents for Piedmont soils, with special emphasis on the application to secondary road projects. Included in the study are laboratory investigations using representative samples of Piedmont soils and the construction of an experimental road with soil-cement and lime-stabilized earth bases. This paper presents results of the laboratory investigations and information from the experimental road.

EFFECTIVENESS OF STABILIZING AGENTS FOR PIEDMONT SOILS

The subsurface materials in the Piedmont region are mostly residual soils derived from metamorphic and igneous rocks. Based on pedological classification, the Cecil series occupies more than 60 percent of the area in this region. Soil samples for laboratory investigations were obtained from various locations throughout the South Carolina Piedmont. The samples represent a wide range in textural compositions, plasticity, and soil series. Classification data and Atterberg limits of the soils are given in Table 1. Results from X-ray diffraction analysis indicate that the clay minerals in the soils sampled are primarily kaolinites.

Sponsored by Committee on Soil-Portland Cement Stabilization.

TABLE 1

SUMMARY OF COMPRESSIVE STRENGTH TESTS AND RELATED DATA OF STABILIZED SOILS

Site	Raw Soil			Cement-Stabilized Soil		Lime-Stabilized Soil		Phosphoric Acid-Stabilized Soil	
	Liquid Limit	Plasticity Index	AASHO Classification	Content of Cement (percent)	Compressive Strength[a] (psi)	Content of Lime (percent)	Compressive Strength[a] (psi)	Content of Phosphoric Acid (percent)	Compressive Strength[a] (psi)
A	48	9	A-7-5(8)	7	329	3	62	1	—[b]
				9	398	5	75	2	—[b]
				12	465	7	72		
				15	492	10	66		
B		NP	A-2-4	10	253			1, 2	—[b]
E-1	29	12	A-6(7)	8	438	8	82	2	36
E-3	50	22	A-7-6(12)	10	448	8	91	2	142
E-4	48	2	A-5(6)	4	142	4	123	1	—[b]
				6	242	8	194	2	102
				8	474				
				10	699				
				13	1,031				
E-5		NP	A-4(4)	6	406	6	133	1	—[b]
F	42	NP	A-5(3)	8	378	7	65	1	196
G		NP	A-1-b	4	337	4	126	1	—[b]
H	50	11	A-7-5(10)	12	423	8	117	1	—[b]
I	38	16	A-6(7)	6	357	6	110	1	102
J	44	9	A-5(3)	4	383	4	171	1	47
K	46	15	A-7-5(8)	8	392	8	149	½, 1	—[b]
L	62	25	A-7-5(15)	8	365	8	149	½, 1	—[b]
M		NP	A-1-b	8	355	8	100	½, 1	—[b]
N	46	17	A-7-6(9)	11	546	8	205	½, 1	—[b]
O	36	16	A-6(7)	4	328	4	83	1	41
P	50	13	A-7-5(2)	6	468	6	214	1	60
Q	65	17	A-7-5(12)	6	310	6	117	½, 1	—[b]
R	40	12	A-6(4)	6	342	6	101	½, 1	—[b]
S	49	6	A-5(1)	6	318	6	123	½, 1	—[b]
T	63	31	A-7-5(18)	12	337	4	24	½, 1	—[b]
U	42	12	A-7-5(7)	8	355	4	23	1	24
V	37	4	A-4(1)	6	400	4	143	½, 1	—[b]
W		NP	A-2-4	6	315	6	108	½, 1	—[b]

[a]Mostly the average of those obtained from tests of 3 specimens.

[b]Because of slacking after immersion, the specimens could not be used for compressive strength tests.

Portland cement, hydrated lime, and phosphoric acid were used as the stabilizing agents in the laboratory experiments with Piedmont soils. The experiments for portland cement stabilization of the soils were made primarily with Type I cement. The hydrated lime used in the laboratory experiments contains approximately 73 percent CaO; the phosphoric acid used contains 75 percent H_3PO_4. The latter was manufactured by electric furnace process.

The effectiveness of the stabilizing agents for the Piedmont soils was evaluated on the basis of compressive strength of the stabilized soils. In this study, the compressive strength serves only as an index representing the influence of a stabilizing agent on the behavior of the raw soil. Procedures for conducting the compressive strength tests are essentially the same as those published by the Portland Cement Association (1). Specimens 2 in. in diameter and approximately 2 in. in height were prepared with a density approaching that obtained by the AASHO standard compaction method T-99. After 7 days of curing in a high humidity cabinet and 24 hours of immersion in water, the specimens were subjected to unconfined compression tests with a rate of deformation of 0.02 in./minute.

The compressive strength test data of soils stabilized with Type I portland cement, hydrated lime, and phosphoric acid are also given in Table 1. The content of all stabilizing agents given in the table is expressed in percentage by weight of oven-dry soil. In the tests using portland cement or hydrated lime as the stabilizing agent, various contents of stabilizing agents were used with soils from all sites. Table 1, however, gives only the data from tests with a single content of stabilizing agent for all soils except those from sites A and E-4. The data of the stabilized soils related to these sites are given as typical examples of the effect on compressive strength due to variations

in the content of stabilizing agents. Further information concerning the laboratory investigations is available in a separate report (2). The following are brief discussions regarding the relative effectiveness of the 3 stabilizing agents for the Piedmont soils.

Stabilization with Portland Cement

The relatively high compressive strength of soil-cement specimens given in Table 1 indicates that portland cement is a more effective stabilizing agent for the Piedmont soils in South Carolina than hydrated lime or phosphoric acid. A substantial reduction in the plasticity of soils upon the addition of portland cement was noted from results of accompanying tests with cohesive soils. The mechanism of cement stabilization of soils has been investigated by Herzog and Mitchell (3), Moh (4), and Noble (5). Information presented in their reports, as well as in other publications related to this subject, indicates that the amount of portland cement required for the stabilization of a given soil is dependent on the mineralogical composition of the soil. The comparatively low cement requirements for the Piedmont soils as indicated later in this paper appear to be related to the fact that the primary constituents in the clay fraction of these soils are kaolinites.

Stabilization with Hydrated Lime

The compressive strength data indicate that the use of hydrated lime as a stabilizing agent results in moderate compressive strength of the stabilized soils. The mechanism of lime stabilization has been studied by many investigators including Diamond and Kinter (6); Eades, Nichols, and Grim (7); and Davidson and Handy (8). Although the complex reactions in soil-lime mixtures are not clearly understood, the information and hypotheses given in these references lead one to believe that lime stabilization of the soils investigated is primarily due to the reaction of lime with siliceous and possibly also aluminous minerals in the soils. In the case of highly micaceous soils, the addition of lime probably results in the formation of abundant cementitious materials identified as calcium silicate hydrates by Eades, Nichols, and Grim (7).

The amount of lime required for adequate stabilization of a given soil may be determined according to the compressive strength, the reduction in plasticity index, or the CBR test data of soil-lime mixtures as reported by Thompson (9). In addition, lime requirements for some soils may be estimated by simple pH tests according to a procedure developed by Eades and Grim (10). Among these methods, the compressive strength test approach is believed to be most suitable for determining the lime requirements of Piedmont soils.

Stabilization with Phosphoric Acid

The data given in Table 1 reveal that phosphoric acid is less effective in stabilizing the Piedmont soils than either portland cement or hydrated lime. Although contents of the stabilizing agent other than the values given in the table were tried in testing some of the soils, the results obtained did not alter this conclusion. Demirel and Davidson (11) reported that the reaction of phosphoric acid with kaolinite was found to be rather slow and incomplete. In view of the fact that the Piedmont soils tested contain primarily kaolinites in the clay fraction, the unfavorable response of the Piedmont soils to this stabilizing agent appears to be essentially due to the relatively slow and incomplete reaction mentioned earlier.

CEMENT REQUIREMENTS OF PIEDMONT SOILS

Although portland cement is effective in stabilizing all soils investigated, appreciable variations are expected in the amount of cement required for adequate stabilization. Table 2 gives a comparison of the cement requirements for all samples determined by various procedures. Although the use of freezing-thawing and wetting-drying tests (12) is generally recognized as a reliable method for determining cement requirements, the test procedures are rather time-consuming. Because the PCA shortcut procedures (1) are applicable only for sandy soils, there is a need for relatively

TABLE 2

CEMENT REQUIREMENTS OF PIEDMONT SOILS

Site	Cement Requirement Determined by Laboratory Testing (percent)			Estimated Cement Content for Setting Up Trial Mixtures (percent)	
	PCA Short-Cut Procedure For Sandy Soils	Freeze-Thaw and Wet-Dry Tests	Minimum 300-psi Compressive Strength	PCA Handbook Procedure	Empirical Equation for Piedmont Soils
A	Not applicable	7	7	14	7
B	10	12	More than 10	—[a]	6
E-1	Not applicable	Not conducted	7	9	7
E-3	Not applicable	10	9	13	9
E-4	Not applicable	5	7	11	6
E-5	4	Not conducted	5	10	6
F	Not applicable	Not conducted	7	10	6
G	4	Not conducted	4	10	6
H	Not applicable	Not conducted	10	15	8
I	Not applicable	Not conducted	6	9	7
J	Slightly less than 4	Not conducted	Less than 4	9	6
K	Not applicable	Not conducted	7	13	7
L	Not applicable	Not conducted	7	15	11
M	5	Not conducted	7	9	6
N	Not applicable	Not conducted	9	11	8
O	Not applicable	Not conducted	4	8	7
P	Not applicable	Not conducted	Less than 6	10	6
Q	Not applicable	Not conducted	6	14	9
R	Not applicable	Not conducted	6	9	6
S	Not applicable	Not conducted	6	10	6
T	Not applicable	Not conducted	12	16	12
U	Not applicable	Not conducted	7	11	7
V	Not applicable	Not conducted	5	8	6
W	6	Not conducted	6	8	6
X-1[b]	6	8	6	10	6
X-2[b]	Not applicable	8	6	12	7
X-3[b]	Not applicable	7	7	12	8
X-4[b]	6	6	7	9	6

[a]Because of extremely low density, the cement requirement of the soil from site B cannot be estimated according to the PCA data of B- and C-horizon soils.

[b]Located at stations 45+00, 53+00, 84+00, and 104+00 respectively along the experimental road described elsewhere in this paper.

short and simple tests applicable to silty and clayey soils or, preferably, to both coarse and fine-grained soils.

Studies for the aforementioned purpose have been made by many investigators including Kemahlioglu, Higgins, and Adam (13) and George and Davidson (14). The use of compressive strength tests for determining the cement requirements of soils in California and, separately, in England was reported by Hveem and Zube (15) and Maclean and Lewis (16) respectively. In this study, an attempt was also made to develop a simplified test procedure for determining the cement requirements of Piedmont soils. Because of the similarities with respect to the geologic origin of Piedmont soils, it was conceived that a design criterion might be established on the basis of the compressive strength of stabilized soils regardless of sandy, silty, or clayey texture. In this respect, a minimum 7-day compressive strength of 300 psi was selected as a tentative criterion for soil-cement to be used in base construction for secondary roads. The cement requirements determined according to this criterion are also given in Table 2. Although this procedure appears to be suitable for determining the cement requirements of Piedmont soils in South Carolina, the reliability of this approach and the adequacy of the tentative criterion require verification by field experiments with soil-cement bases and continual observation of their performance. Field experiments for this purpose are described elsewhere in this paper.

In the design of soil-cement mixtures, it is desirable to obtain approximate estimates of cement requirements to assist in setting up trial soil-cement mixtures for laboratory tests. This may be achieved by a method developed by Diamond and Kinter (17) or according to the PCA procedure (1, pp. 13-14). The first method is applicable for plastic soils only. To estimate the cement requirements of all soils used in this

study, the PCA procedure was followed. Results obtained by this method (Table 2) indicate that the actual cement requirements determined by previously described procedures are consistently lower than the estimated values. This is apparently due to the fact that the PCA method represents a general procedure formulated on the basis of information from all kinds of soil located in a wide variety of geologic regions. For soils with similar mineralogical composition and located within a particular geologic region, it is believed possible to develop a specific method that may be used for improving the accuracy of cement requirement estimates. To this end, efforts were made to formulate a procedure for estimating the cement requirements of Piedmont soils. After an analysis of the results from laboratory investigations, the following empirical equation was obtained:

$$\text{Cement content (percent)} = 3 + (\text{group index of soil}/2)$$

$$\text{Minimum cement content} = 6 \text{ percent}$$

The cement contents computed from this equation are given in Table 2. The data indicate that the cement contents estimated by the empirical equation are (a) appreciably lower than those estimated by the PCA method and (b) fairly close to the actual cement requirements of Piedmont soils. In view of the fact that kaolinites are the predominant type of clay mineral in the Piedmont soils, it can be expected that the cement requirements of these subsurface materials would be lower than those of soils having comparable clay fraction but containing montmorillonites as the predominant type of clay mineral.

STABILIZED EARTH BASES IN EXPERIMENTAL ROAD

For the study of construction procedures and performance of stabilized earth bases, an experimental road was constructed along route S-671, a secondary road in Greenville County, South Carolina. Type I portland cement and hydrated lime were used as stabilizing agents for the experimental bases. The subsurface materials at the selected site are representative of the Piedmont soils in South Carolina.

Layout and Construction of Experimental Bases

As discussed previously, portland cement was found to be the most effective stabilizing agent for the Piedmont soils, and hydrated lime was rated second. The experimental road was, therefore, planned in such a manner that the major emphasis was placed on soil-cement bases. In the layout of the experimental sections, the amount of stabilizing agent to be mixed with the soils was selected mainly on the basis of the compressive strength of soil-cement and soil-lime mixtures. For soil-cement bases, considerations were also given to the results of freezing-thawing and wetting-drying tests. Although a uniform thickness of 5 in. was adopted for the soil-lime bases, sections of soil-cement bases 4 and 6 in. thick were used for evaluating the thickness effect on their performance. Figure 1 shows the detailed layout of all experimental sections.

In selecting the equipment for the construction of the stabilized earth bases, it was recognized that the use of stationary mixing plants or single-pass traveling mixing machines would result in more uniform mixtures and better construction control than the use of multipass rotary mixers. Nevertheless, multipass rotary mixers together with motor graders were actually used because of the intent of evaluating the type of equipment and procedure that can be expected in the construction of secondary roads in South Carolina.

The application of stabilizing agents was done by placing bags of portland cement or hydrated lime along the roadway and then spreading the cement and lime over the soil to be stabilized. The degree of pulverization of the raw soils required in the South Carolina standard specifications is that 100 percent by dry weight pass 1-in. sieve and a minimum of 80 percent pass the No. 4 sieve, exclusive of gravel and stone retained on the sieve. All stabilized earth bases were compacted at or near the optimum mois-

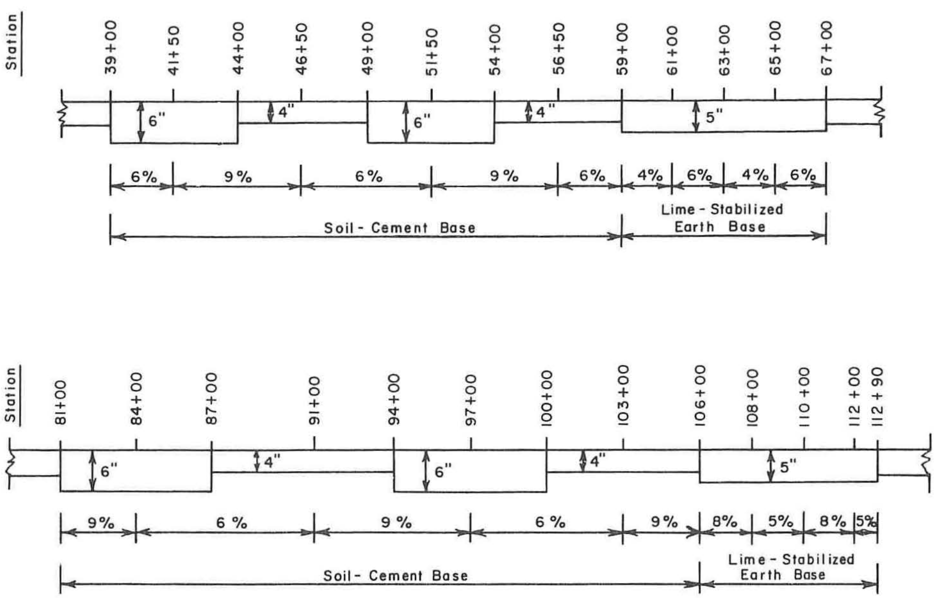

Figure 1. Layout of stabilized earth bases along experimental road.

ture content. For soil-cement bases, the specification requires that compaction of the base should be completed within 6 hours after application of water to the mixture of soil and cement. For soil-lime bases, the moist soil-lime mixture was allowed to age for 48 to 72 hours before final mixing and compaction. During the aging period, the roadway surface was sealed by light compaction to prevent excessive amounts of water from percolating into the mixture in the event of rain.

After final compaction, the stabilized earth bases were primed with MC-30 cutback asphalt for maintaining the proper moisture content during curing. When all experimental sections were completed, they were covered with a bituminous surfacing according to the procedures indicated in the South Carolina standard specifications for "double treatment" using emulsified asphalt as the binder. The width of stabilized earth bases is 23 ft and that of the bituminous surfacing is 22 ft. The experimental bases were constructed in July 1967 when the weather was generally dry and hot.

Properties of Raw and Stabilized Soils

The AASHO classification, liquid limit, and plasticity index of soils sampled after the final grading of the experimental road are given in Table 3. The soils listed in this table are from either B- or C-horizon. In the area where the experimental road is located, the materials from B-horizon are usually A-6 or A-7 soils; those from C-horizon are often A-2, A-4, or A-5 soils. Also given in Table 3 are the liquid limit and plasticity index of the stabilized soils. Test samples were taken just before compaction of the stabilized earth bases and on testing were found to contain the percentage of stabilizing agent given in the table. As expected, the data given in the table indicate a reduction in the plasticity index of the soils due to the addition of stabilizing agents.

The stress-strain characteristics of the raw and stabilized soils were evaluated by conducting triaxial tests using samples representative of the subsurface materials along the experimental road. The samples selected for this purpose are the A-7-5(9) soil from B-horizon at station 84+00 and the A-2-4 soil from C-horizon at station 110+00. All specimens were compacted in such a manner as to provide the anticipated field density. Specimens of raw soils were subjected to capillary absorption before being used for triaxial tests. In the case of stabilized soils, the test specimens were cured for 1 week, subjected to capillary absorption, and then used for triaxial tests.

TABLE 3

CLASSIFICATION OF SOILS ALONG EXPERIMENTAL ROAD AND EFFECT OF STABILIZING AGENTS ON PLASTICITY

Station	Raw Soil			Stabilized Soil		
	AASHO Classification	Liquid Limit	Plasticity Index	Stabilizing Agent (percent)	Liquid Limit[a]	Plasticity Index[a]
40+00	A-7-5(9)	49	18	7.4 cement	37	4
43+00	A-7-5(8)	44	14	14.2 cement	38	1
45+00	A-4(2)	40	4	6.3 cement	Not conducted	Not conducted
48+00	A-7-6(7)	45	16	5.2 cement	34	3
50+00	A-4(5)	36	9	10.4 cement		NP
53+00	A-7-5(3)	43	11	5.7 cement		NP
55+00	A-7-6(10)	49	21	11.2 cement		NP
58+00	A-7-6(6)	45	17	7.9 cement		NP
60+00	A-4(3)	35	7	4.9 lime	Not conducted	Not conducted
62+00	A-4(1)	32	10	5.2 lime		NP
64+00	A-2-4(0)	36	1	4.3 lime		NP
66+00	A-6(3)	38	14	4.5 lime		NP
82+00	A-6(3)	35	11	6.0 cement	34	2
86+00	A-7-6(10)	49	20	3.8 cement	36	8
88+00	A-7-5(9)	47	17	5.5 cement		NP
92+00	A-7-6(10)	47	19	9.0 cement		NP
99+00	A-7-6(7)	44	16	5.5 cement		NP
102+00	A-6(4)	37	11	5.5 cement		NP
104+00	A-6(3)	37	11	7.5 cement		NP
107+00	A-7-5(4)	47	11	6.3 lime		NP
109+00	A-2-4		NP	4.3 lime	Not conducted	Not conducted
111+00	A-4(0)	38	9	6.3 lime		NP

[a]Atterberg limits tests for the stabilized soils were made approximately one month after the soils had been mixed with the stabilizing agents.

The moisture content, dry density, and modulus of deformation data for all materials tested are given in Table 4. The effectiveness of stabilizing agents is indicated by the extremely high moduli of deformation of stabilized soils in comparison with those of the raw soils.

TABLE 4

TRIAXIAL TEST DATA OF RAW AND STABILIZED SOILS

Station	Soil	Moisture Content During Molding (percent)	Dry Density (pcf)	Modulus of Deformation[a] (psi)
84+00	Raw	18.6	97.2	1,100
	Stabilized with 6 percent cement	20.3	96.8	71,400
	Stabilized with 9 percent cement	20.6	96.4	166,700
	Stabilized with 6 percent lime	24.5	95.2	15,000
	Stabilized with 9 percent lime	23.6	93.9	35,000
110+00	Raw	17.1	103.9	1,460
	Stabilized with 5 percent cement	16.9	103.9	28,600
	Stabilized with 6 percent cement	16.8	105.3	50,000
	Stabilized with 5 percent lime	18.2	102.2	28,790
	Stabilized with 8 percent lime	17.8	101.0	35,540

Note: Triaxial tests were conducted under unconsolidated undrained conditions with test procedures as described by Chu, Humphries, and Fletcher (20).

[a]With a continuing pressure of 10 psi and a deviator stress of 5 psi; data given are the average of similar specimens.

TABLE 5

COMPARISON OF LABORATORY AND FIELD COMPACTION OF STABILIZED SOILS

Stabilizing Agent	Station	Laboratory Compaction: Maximum Dry Density (pcf)	Laboratory Compaction: Optimum Moisture Content (percent)	Field Compaction: Actual Dry Density (pcf)	Field Compaction: Actual Moisture Content (percent)	Percent Compaction
Cement	40+00	102.1	20.1	100.2	16.9	98.1
	43+00	102.1	20.1	98.8	19.4	96.8
	45+00	111.6	18.0	97.0	19.4	86.9
	48+00	102.1	20.1	102.2	20.8	100.0
	50+00	111.6	18.0	97.0	19.6	86.9
	53+00	102.1	20.1	97.2	20.5	95.2
	55+00	102.1	20.1	98.9	21.2	96.9
	58+00	102.1	20.1	99.6	20.8	97.6
	82+00	112.2	16.0	106.3	15.6	94.7
	86+00	102.1	20.1	102.8	20.7	100.7
	88+00	102.1	20.1	111.7	18.0	109.4
	92+00	102.1	20.1	102.8	17.1	100.7
	95+50	111.6	18.0	96.0	16.1	86.0
	96+50	111.6	18.0	101.3	18.7	90.8
	99+00	102.1	20.1	105.2	18.9	103.0
	102+00	109.5	18.5	109.0	19.1	99.5
	104+00	109.5	18.5	109.1	17.8	99.6
Lime	60+00	101.8	21.0	92.1	17.1	90.5
	62+00	101.8	21.0	98.9	19.4	97.2
	64+00	106.3	17.5	94.6	19.8	89.0
	66+00	104.0	22.0	99.9	15.9	96.1
	107+00	103.0	22.0	103.5	21.4	100.5
	109+00	107.0	18.3	102.0	19.4	95.3
	111+00	101.8	21.0	98.5	19.4	96.8
	112+50	101.8	21.0	98.4	18.6	96.7

Compaction of Stabilized Soils

The maximum density of stabilized soils was determined in the laboratory according to AASHO T-134 test procedures (12). In the field, the soil-cement and soil-lime bases were compacted by pneumatic rollers. After compaction, field density tests were performed at selected locations. The density and moisture content data from laboratory and field tests are given in Table 5. The percentage of compaction of stabilized earth bases may be influenced by many factors including the moisture content during compaction and the lift thickness. Data related to the optimum and field moisture contents of the stabilized soils are given in Table 5. Figure 2 shows that at most locations the actual thickness, as determined by the height of cored specimens obtained from the soil-

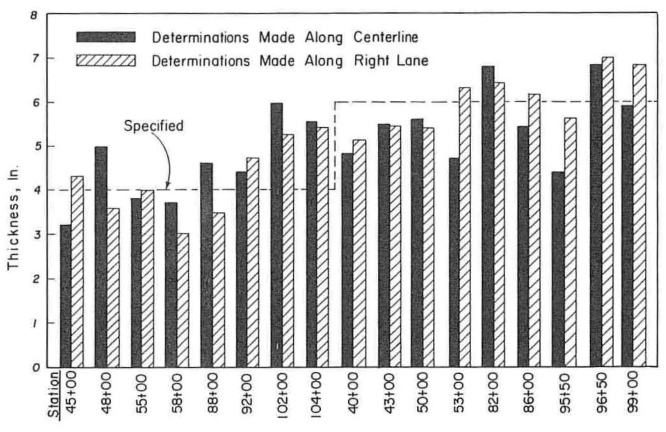

Figure 2. Thickness of soil-cement bases.

cement bases, differs by less than 1 in. from the specified value. The percentage of compaction given in Table 5 is also influenced by the possible difference between the stabilized soil used for the laboratory compaction test and that actually encountered at a particular location. The extremely low values of percentage of compaction at stations 45+00, 50+00, and 95+50 and the unusually high value at station 88+00 are probably due to the aforementioned factor.

Compressive Strength of Stabilized Soils

Compressive strength tests were conducted by using laboratory prepared specimens of stabilized soils as well as cored specimens representing the experimental bases. The laboratory specimens were compacted by a drop hammer with a compactive effort comparable to that specified in the AASHO T-99 compaction test. After compaction, specimens were cured for 7 days and immersed in water for 24 hours before the compressive strength test. Although the cored specimens were also immersed for 24 hours before the test, the time between compaction and testing is much longer for cored specimens than it is for laboratory specimens. Compressive strength test data related to the experimental bases are given in Table 6. The tabulated compressive strength is the average of values from tests of 2 similar specimens. Also given in the table are the ratios of compressive strength of cored specimens to that of laboratory prepared specimens.

Data given in Table 6 indicate that, for soil-cement bases, the compressive strength of cored specimens obtained in August 1967, approximately 1 month after the construction of the experimental bases, is substantially lower than that of laboratory specimens. This is apparently due to differences between field and laboratory conditions with respect to the extent of pulverization, uniformity in spreading or applying cement, degree of mixing, effectiveness of compaction, and efficiency of curing. In reviewing these various factors, it is noted that the effects related to the methods for pulverization,

TABLE 6

COMPARISON OF COMPRESSIVE STRENGTH OF LABORATORY PREPARED AND CORED SPECIMENS OF STABILIZED SOILS

Stabilizing Agent	Station	Compressive Strength of Laboratory Prepared Specimens, q_0[a] (psi)	Compressive Strength of Cored Specimens[b] (psi)			Ratio of Compressive Strength of Cored Specimens to Laboratory Prepared Specimens		
			q_1	q_2	q_3	q_1/q_0	q_2/q_0	q_3/q_0
Cement	40+00	270	107	157	219	0.4	0.6	0.8
	43+00	344	97	190	227	0.3	0.6	0.7
	45+00	429	209	—c	—c	0.5	—	—
	48+00	256	121	235	—c	0.5	0.9	—
	50+00	255	144	165	—c	0.6	0.7	—
	53+00	330	217	398	577	0.7	1.2	1.8
	55+00	374	112	353	—c	0.3	0.9	—
	58+00	269	133	358	—c	0.5	1.3	—
	82+00	526	147	184	195	0.3	0.4	0.4
	86+00	183	118	153	330	0.7	0.8	1.8
	92+00	314	138	399	430	0.4	1.3	1.4
	99+00	275	115	203	298	0.4	0.7	1.1
	102+00	245	114	274	466	0.5	1.1	1.9
Lime	60+00	102	—c	—c	—c	—	—	—
	62+00	28	—c	—c	—c	—	—	—
	64+00	123	—c	65	—c	—	—	—
	66+00	124	—c	—c	—c	—	—	—
	107+00	195	218	288	486	1.1	1.5	2.5
	109+00	290	180	281	—c	0.6	1.0	—
	111+00	154	149	—c	—c	1.0	—	—
	112+50	115	68	60	—c	0.6	0.5	—

Note: Average of the ratios is q_1/q_0, 0.5; q_2/q_0, 0.9; and q_3/q_0, 1.2.

[a]Laboratory prepared specimens were cured for 7 days before testing. The contents of stabilizing agents are shown in Figure 1.
[b]Cored specimens q_1, q_2, and q_3 were obtained in August 1967, October 1968, and December 1969 respectively. The contents of stabilizing agents in cored specimens are given in Table 7.
[c]Specimens not satisfactory for compressive strength tests.

cement spreading, and mixing are of special significance in this study. A brief discussion in regard to these effects follows.

The actual cement contents in soil-cement bases were determined by laboratory tests using cored specimens in accordance with procedures of AASHO Test Method T-144-57 (12). A comparison of the specified and actual cement contents of the soil-cement bases as given in Table 7 indicates that there are substantial variations in actual cement contents within each experimental section. Although the accuracy of laboratory tests for determining the cement contents may have some influence on test results, the major factors causing the great variation in cement contents appear to be related to the equipment and procedure of construction. As discussed previously, the experimental bases were constructed by the type of equipment and procedure that can be expected in secondary road construction in South Carolina. Consequently, the uniformity of mixture within each experimental section and the degree of mixing are expected to be inferior to those obtainable by the use of stationary mixing plants or single-pass traveling mixing machines. Based on a laboratory study of soil-cement mixtures, Baker (18) reported the pronounced effect on compressive strength due to variations in the degree of mixing. According to the laboratory data and other information given here, it is believed that the adverse effects related to uniformity of mixture and degree of mixing are major causes for the relatively low compressive strength of the cored specimens taken in 1967.

The data obtained from the cored specimens of soil-cement bases taken in 1967, 1968, and 1969 (Table 6) show a general increase in compressive strength with time. As a result, the compressive strength of cored specimens taken approximately 2½ years after construction is higher than that of laboratory prepared specimens tested 7 days after compaction. The rate of increase in compressive strength in the post-construction period is indicated by the average of the ratios of the compressive strength of cored specimens to that of laboratory prepared specimens as noted in the table. The

TABLE 7

COMPARISON OF SPECIFIED AND ACTUAL CONTENTS OF STABILIZING AGENTS IN EXPERIMENTAL BASES

Stabilizing Agent	Station	Specified Percentage of Stabilizing Agent	Actual Percentage of Stabilizing Agent Determined From Core					
			August 1967		October 1968		December 1969	
			Core 1	Core 2	Core 1	Core 2	Core 1	Core 2
Cement	40+00	6	8.1	5.2	7.0	4.9	—[a]	4.6
	43+00	9	8.7	11.2	10.4	7.9	7.4	9.6
	45+00	9	6.5	10.2	6.0	9.0	7.1	8.0
	48+00	6	—[a]	4.8	6.0	8.4	5.0	9.3
	50+00	6	—[a]	7.1	9.1	8.6	9.1	5.8
	53+00	9	—[a]	12.6	10.4	8.4	9.3	18.2
	55+00	9	—[a]	7.0	10.1	9.7	6.9	8.6
	58+00	6	6.8	10.6	7.5	8.6	7.1	9.0
	82+00	9	—[a]	9.1	4.9	7.4	4.8	6.3
	86+00	6	5.5	5.2	4.0	6.7	10.8	6.5
	88+00	6	4.9	4.3	5.4	5.8	4.9	3.8
	92+00	9	7.2	6.3	6.5	6.7	7.2	5.7
	95+50	9	7.5	10.0	6.9	8.2	8.0	8.6
	96+50	9	6.3	7.2	7.1	8.6	6.4	9.1
	99+00	6	—[a]	13.4	5.6	5.9	5.1	5.7
	102+00	6	—[a]	5.0	4.8	4.1	3.4	4.9
	104+00	9	7.2	5.6	7.1	6.6	4.9	4.3
Lime	60+00	4	—[a]	2.9	7.6	—[a]	7.2	4.5
	62+00	6	—[a]	5.6	5.9	6.8	2.2	—[a]
	64+00	4	—[a]	3.8	5.4	3.3	4.1	2.9
	66+00	6	—[a]	4.1	3.3	3.5	3.8	4.3
	107+00	8	6.8	5.6	7.7	6.7	6.5	5.9
	109+00	5	—[a]	4.2	5.3	6.5	5.2	3.9
	111+00	8	6.1	6.5	7.3	6.7	8.1	6.6
	112+50	5	4.1	4.3	4.3	5.0	4.5	1.6

[a]Tests for determining the content of stabilizing agent were not conducted at the locations indicated.

increase in compressive strength with time is another indication of the effectiveness of portland cement for the stabilization of the soils investigated.

In the case of soil-lime bases, the findings concerning substantial variations in lime contents and the general increase in compressive strength with time are similar to those regarding soil-cement bases discussed earlier. It will be noted, however, that information from the soil-lime bases is rather limited because of difficulties in obtaining satisfactory specimens for compressive strength tests.

Performance of Experimental Road

After completion of the stabilized earth bases and bituminous surfacing in the summer of 1967, traffic counts were conducted to determine the number of vehicles traveling on the experimental road. The average daily traffic during the past 3 years varied from 120 to 250 vehicles of which most were passenger cars. The traffic volume indicated is slightly higher than the average for secondary roads in South Carolina.

The performance of the experimental road has been studied by frequent inspections of the conditions in various experimental sections. The findings from the inspections made in June 1968 and July 1970 are given in Table 8. In general, the soil-cement bases were found to provide satisfactory performance except for some edge raveling as a result of shoulder erosion. The general conditions of the soil-lime bases, however, are less favorable than those of the soil-cement bases.

To assist in the evaluation of the performance of various experimental sections, Benkelman beam deflection measurements were made periodically at selected locations. In all measurements, the applied load is 18 kips on single axle and the tire pressure is 80 psi. The "normal procedure" as described by Benkelman, Kingham, and Fang (19) was followed in conducting the measurements. Data from the deflection measurements made in 1968, 1969, and 1970 are shown in Figures 3 and 4. The deflection at each location shown in the figures is the average of the values from 2 separate measurements conducted along the outer wheelpath in adjacent areas.

TABLE 8

GENERAL CONDITIONS OF EXPERIMENTAL SECTIONS

Stabilizing Agent	Station	Conditions as Observed in June 1968 (1 year after construction)	Conditions as Observed in July 1970 (3 years after construction)
Cement	39+00 to 59+00	No rutting or surface cracking except that erosion of shoulder resulted in some cracking at north side edge of bituminous surfacing in area between station 54+00 and 57+00.	Road in good condition except that erosion of shoulder has resulted in further cracking along edge of bituminous surfacing. Patching along edge has been done in some areas.
Lime	59+00 to 67+00	Some rutting along wheelpath. Rut depth up to $^3/_4$ in. Extensive raveling at edges of stabilized base.	Conditions are inferior to those in soil-cement section between stations 54+00 and 57+00. Some areas have been repaired by patching, but cracking and extrusion of fine materials through cracks are observed.
Cement	81+00 to 106+00	No rutting or surface cracking except some edge raveling in vicinity of station 91+00 and in area between station 97+00 and 106+00. Edge raveling in vicinity of station 91+00 apparently because soil-cement base is approximately 1 ft inside edge of bituminous surfacing.	Conditions generally similar to those in soil-cement section from station 39+00 to 59+00 except that occasional longitudinal cracking is observed at approximately one-third of width of stabilized base.
Lime	106+00 to 112+90	No rutting or surface cracking except some edge raveling.	Edge raveling in some areas has been repaired by patching. Cracking occurred along wheelpath. In general, conditions are similar to those of other lime-stabilized base section.

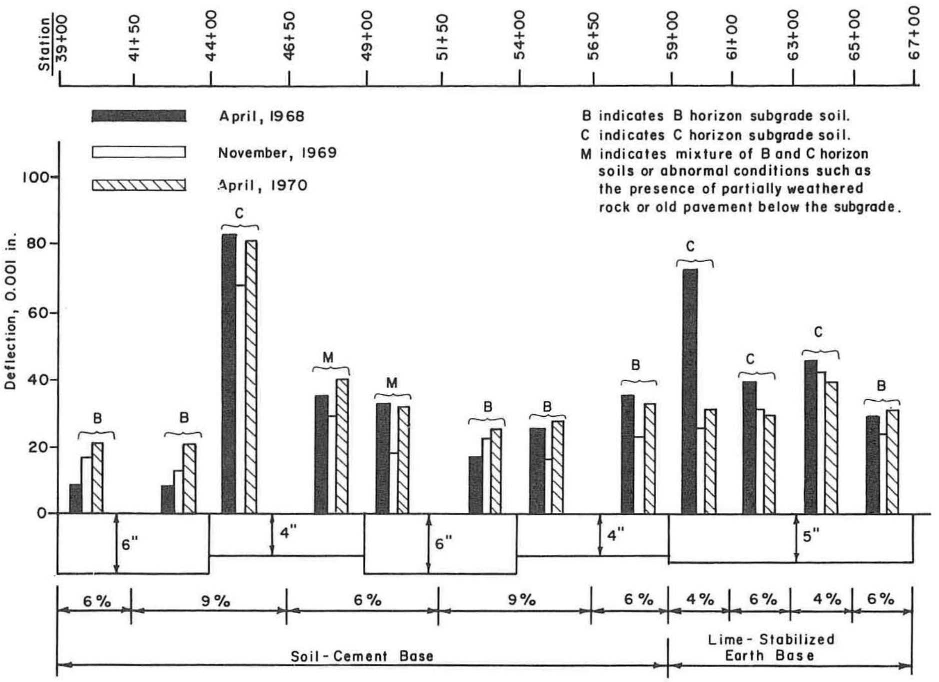

Figure 3. Benkelman beam deflections on experimental road between stations 39+00 and 67+00.

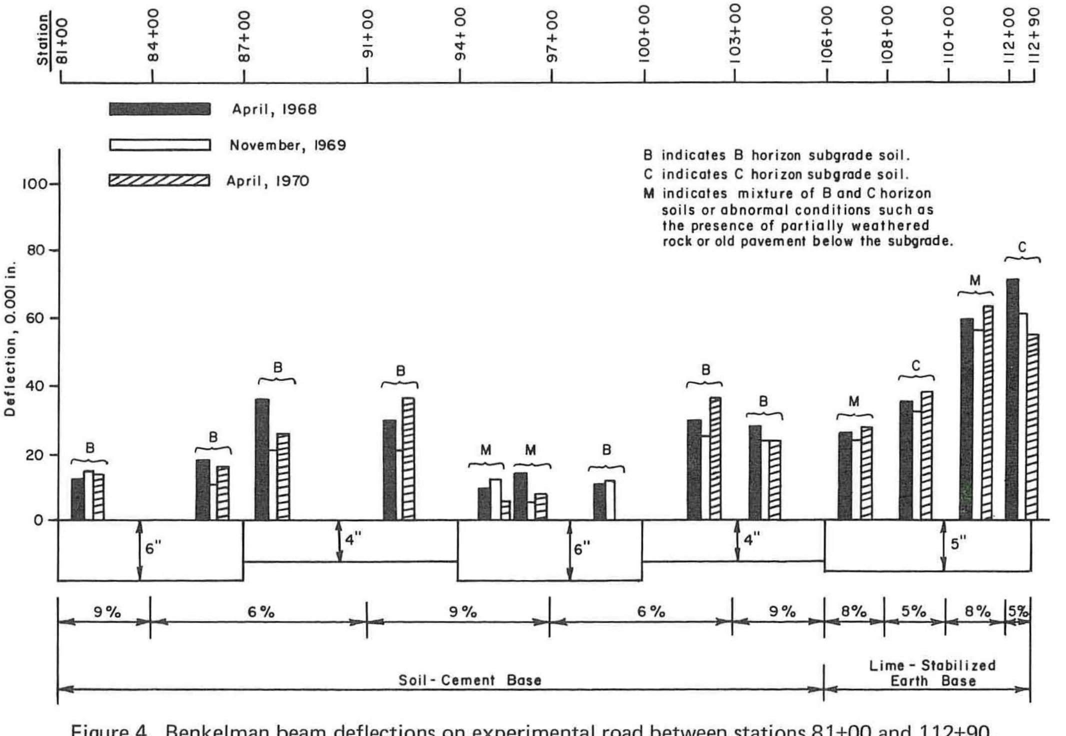

Figure 4. Benkelman beam deflections on experimental road between stations 81+00 and 112+90.

The deflection of a pavement under an applied wheel load is dependent on many variables such as the thickness of the pavement system and the stress-strain characteristics of both the subgrade and the pavement components. Variations in temperature of the pavement and moisture content of the subgrade soil may also affect pavement deflection because of their influence on the aforementioned variables. In this study, the effect related to pavement temperature and subgrade moisture is believed to be of minor importance. The other variables indicated, however, may all have significant influence on the deflection values shown in Figures 3 and 4. Because the experimental road was planned for several objectives and not specifically for evaluating the individual effect of each variable, it is difficult to formulate specific conclusions on the basis of the deflection data. Following is a general discussion in connection with the results obtained from the deflection study.

As noted previously, C-horizon soils along the experimental road are the typical micaceous A-2, A-4, or A-5 soils existing in the Piedmont region. The moduli of elasticity of these soils are often found to be extremely low. This is probably the main reason for the relatively high deflections at locations having C-horizon subgrade soils under the experimental bases. In regard to the performance of the stabilized earth bases, the deflection data shown in Figures 3 and 4, together with the information given in Table 8, indicate that there has been no significant change in the serviceability of the experimental road, especially the soil-cement sections, in the 3-year post-construction period.

Among the objectives of the experimental road are the evaluation of thickness effect of soil-cement bases and verification of the tentative 300-psi minimum compressive strength requirement for designing soil-cement mixtures to be used as base materials on secondary road projects. Information available at this time is inadequate for the desired purposes. Continued observation and measurements have been planned for obtaining additional information from the experimental road. As to the minimum compressive strength requirement, the performance study described and the laboratory test results given in Table 2 appear to have provided sufficient information to suggest that the application of the tentative design criterion be continued at least for the time being.

SUMMARY

1. Based on laboratory investigations conducted in this study, portland cement is the most effective stabilizing agent for the Piedmont soils in South Carolina and hydrated lime is rated second. The performance of soil-cement bases on the experimental road is also superior to that of soil-lime bases.

2. The experimental road was constructed by using the type of equipment and procedure that can be expected to be used on secondary road projects. As a result, the compressive strength of soil-cement bases represented by cored specimens is lower than the 7-day compressive strength of laboratory prepared specimens until approximately $2\frac{1}{2}$ years after construction. At that time, the compressive strength of soil-cement bases exceeded that of laboratory specimens.

3. A 7-day compressive strength of 300 psi was tentatively established as the minimum requirement for designing soil-cement mixtures to be used as base materials on secondary road projects in the South Carolina Piedmont. Although no definite conclusions can be drawn at this time, the field and laboratory investigations in this study are believed to have provided sufficient information to suggest that application of the tentative design criterion be continued at least for the time being.

4. Approximate cement requirements of Piedmont soils may be estimated by a simplified method formulated on the basis of test results obtained from this study. The approximate estimate of the cement requirement of a given soil is to assist in setting up trial soil-cement mixtures for compressive strength or other laboratory tests with the objective of determining the actual cement requirement.

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EVALUATION AND PREDICTION OF THE TENSILE PROPERTIES OF ASPHALT-TREATED MATERIALS

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The increased use of asphalt-stabilized subbases in rigid pavement structures has created the need for a rational procedure by which to design these subbases. A design procedure based on layered theory is presently under development at the University of Texas at Austin to satisfy this need. This theoretical design method consequently requires that material characterization constants such as modulus of elasticity, Poisson's ratio, and failure strains be estimated for a variety of asphalt-stabilized materials. This paper describes a study that was undertaken to evaluate the effects of 7 different factors on the tensile properties of asphalt-treated materials. The 7 factors investigated include aggregate type, aggregate gradation, asphalt viscosity, asphalt content, mixing temperature, compaction temperature, and curing temperature. The test responses discussed are modulus of elasticity, tensile strength, and total tensile strain. The results reported were obtained from a carefully controlled indirect tensile test. The data from this study indicate that there is no trend or correlation between either modulus of elasticity and density or tensile strength and density. Hence, changes in density alone cannot be used as a measure of changes in tensile properties of asphalt-treated materials but must be accompanied by careful consideration of the factors involved in the mix design. Because of the dominant effect of compaction temperature on the 3 tensile properties, it is recommended that present laboratory test procedures be extended to include the evaluation of the effect of changes in compaction temperature and that closer control of compaction temperature in the field be established through specification requirements.

•THE INCREASED use of asphalt-stabilized materials as subbases for rigid pavements has created the need for a rational design procedure for these highway materials. To satisfy this need, a design procedure based on layered theory is presently under development at the University of Texas at Austin (1). Such a procedure requires that material characterization constants such as modulus of elasticity, Poisson's ratio, and failure strain be known for a variety of highway materials. Estimated values of these tensile properties can be obtained from indirect tensile test results (2, 3, 4).

In previous work (2) the tensile strengths for asphalt-treated materials involving a wide variety of mix variables were evaluated. This previous screening study provided insight into the complexity of the asphalt-stabilization process and indicated that interactions of 2 or more variables are involved. The study, however, had 2 limitations: (a) most of the effects produced by the interaction of 3 or more variables could not be quantified and evaluated; and (b) the nonlinear effects for the variables could not be evaluated.

In addition, because the technique for estimating material characterization constants (3) from the indirect tensile test was not available, this screening experiment was limited primarily to an evaluation of tensile strength. In the development of techniques

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for estimating modulus of elasticity, Poisson's ratio, and failure strains, there were minor changes in the testing equipment and procedures that prevented the estimation of these material constants for the mixtures evaluated in the screening experiment. Thus, a more complete followup study was conducted (5). The primary objectives of this study included (a) the examination of how changes in a number of independent variables affected certain dependent or response variables and (b) the development of a method of predicting the variations in these responses with changes in the independent variables. The first was accomplished by analyses of variance and the second was obtained from regression analyses.

EXPERIMENTAL PROGRAM

This program was designed to investigate 7 different factors considered to affect the tensile properties of asphalt-treated materials. A central composite design (6, 7, 8) was utilized, and it allowed the evaluation of nonlinear effects of 6 of the 7 factors. The factors and levels selected for this investigation are given in Table 1.

Selection of Factors

Two aggregate types that exhibited relatively extreme characteristics were selected. The gravel was a naturally occurring, subrounded, nonporous aggregate with a relatively smooth surface texture. The limestone aggregate, on the other hand, was a naturally occurring, porous aggregate with angular particles and a relatively rough surface texture when crushed.

The gradation curves for the fine, medium, and coarse-graded mixtures are shown in Figure 1. The fine, medium, and coarse gradations were identified by 2-mm, 4-mm, and 6-mm particles. These dimensions represent the diameter of the particle that was larger than 60 percent of the particles in the total mixture.

The temperature-viscosity relationships for the AC-5, AC-10, and AC-20 asphalt-cements are shown in Figure 2. The 3 levels were specified by the slope of the line connecting the viscosity at a temperature of 140 F and the viscosity at a temperature of 275 F, which was determined by the equation

$$\text{Slope} = \frac{\log (V140) - \log (V275)}{\log (140) - \log (275)}$$

TABLE 1

LEVELS OF FACTORS USED IN REGRESSION EQUATIONS

Factor	Classification	Description	Levels Used in Regression Equations
Aggregate type	Qualitative	Crushed limestone	A(-1) = 0
		gravel	A(+1) = 2
Aggregate gradation[a]	Quantitative	Fine	B(-1) = 2
		Medium	B(0) = 4
		Course	B(+1) = 6
Asphalt viscosity[a]	Quantitative	AC-5	C(-1) = 8.5
		AC-10	C(0) = 9.0
		AC-20	C(+1) = 9.7
Asphalt content, percent	Quantitative	Low-low	D(-2) = 4.0
		Low	D(-1) = 5.5
		Medium	D(0) = 7.0
		High	D(+1) = 8.5
		High-high	D(+2) = 10.0
Mixing temperature, deg F	Quantitative	Low	F(-1) = 250
		Medium	F(0) = 300
		High	F(+1) = 350
Compaction temperature, deg F	Quantitative	Low	G(-1) = 200
		Medium	G(0) = 250
		High	G(+1) = 300
Curing temperature, deg F	Quantitative	Low	H(-1) = 40
		Medium	H(0) = 75
		High	H(+1) = 110

[a]See Figures 1 and 2 for method of determining levels.

where V140 and V275 are the viscosities at 140 and 275 F. The AC-5, AC-10, and AC-20 asphalt-cements were identified by slopes of 8.5, 9.0, and 9.7 respectively.

The levels of asphalt content were chosen on the basis of the results of the screening experiment so that an optimum asphalt content could be obtained. Medium levels were also included in this experiment for the remaining quantitative variables. Aggregate type was a qualitative variable, i.e., a variable in which the different levels could not be arranged in order of magnitude (7). Therefore, no medium level was specified for aggregate type.

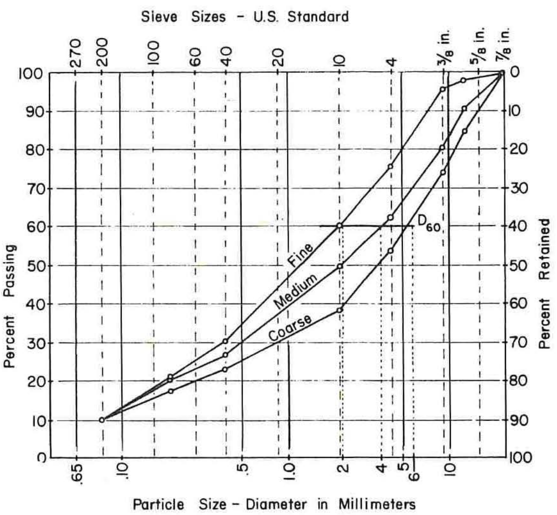

Figure 1. Gradation curves for aggregate mixtures.

Parameters Evaluated

In this study the following variables were evaluated: tensile strength, total tensile strain at failure, and modulus of elasticity. The development of the equations for these 3 variables is presented in another report (3). The value of the modulus of elasticity was obtained from a portion of the load-total deformation curve that was essentially linear.

Preparation and Testing Procedure

All asphalt-treated materials were mixed for 3 minutes and compacted in a Texas gyratory-shear molding press to form a cylindrical specimen with a nominal 4-in. diameter and 2-in. height. Following compaction, the specimens were allowed to cool to room temperature and their densities were determined. Then the specimens were cured for 14 days at the designated curing temperature. At the end of the curing period, the specimens were tested in indirect tension at 75 F and at a loading rate of 2.0 in./min. The test and equipment are described in detail in other reports (2, 3, 4).

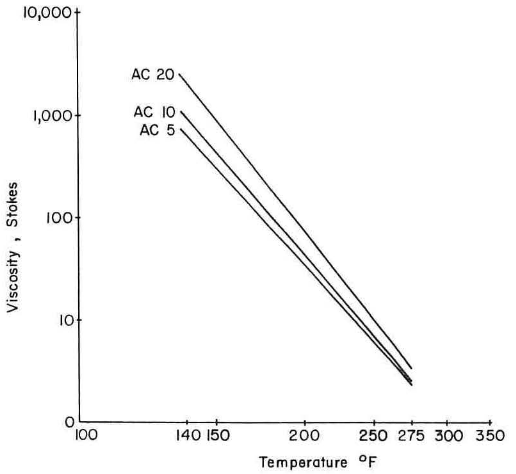

Figure 2. Temperature-viscosity relationship for asphalt cements.

Statistical Design and Analysis

This design consisted of a 2^7 full factorial with 128 possible combinations of the 7 factors, which allowed the analysis of main effects and interaction effects, and 52 wall points and 4 center points that allowed curvilinear effects to be evaluated.

The analysis consisted of an analysis of variance to determine the significance of main effects, interaction effects, and nonlinear effects produced by the 7 independent factors and a regression analysis to develop predictive equations for estimating the tensile properties of asphalt-treated materials for a given set of the 7 independent variables.

ANALYSIS

There were a number of factors and interactions that significantly affected the

tensile strength, tensile failure strain, and modulus of elasticity of asphalt-treated materials; however, not all of these effects had practical significance. In other words, the effect, although measurable, was not large and probably would make no effective difference in the engineering application of the results. The effects judged to have practical meaning corresponded to a probability level of 0.5 percent for tensile strength and modulus of elasticity and 5 percent for total tensile strain. Those factors and their interactions of practical engineering significance to tensile strength, total tensile strain, and modulus of elasticity are given in Tables 2, 3, and 4 respectively. The effects that are listed as B_Q and D_Q were quadratic or nonlinear effects due to gradation and asphalt content respectively.

TABLE 2

ANALYSIS OF VARIANCE FOR TENSILE STRENGTH AT 75 F

Source of Variation[a]	Degree of Freedom	Mean Squares	F-Value	Significance Level (percent)
BD	1	137,615	299.80	0.5
G	1	105,580	230.01	0.5
D	1	59,391	129.39	0.5
D_Q(quadratic)	1	45,525	99.18	0.5
C	1	29,900	65.14	0.5
BDG	1	16,199	35.29	0.5
B_Q(quadratic)	1	15,185	33.08	0.5
ABD	1	14,699	32.02	0.5
B	1	12,368	26.94	0.5
BG	1	6,065	13.21	0.5
Experimental error	79	459.0		

[a]A = aggregate type; B = aggregate gradation; C = asphalt viscosity; D = asphalt content; F = mixing temperature; and G = compaction temperature.

TABLE 3

ANALYSIS OF VARIANCE FOR TENSILE STRAIN AT FAILURE AT 75 F

Source of Variation[a]	Degree of Freedom	Mean Squares ($\times 10^{-6}$)	F-Value	Significance Level (percent)
G	1	3.133	19.06	0.5
AD	1	1.723	10.48	0.5
D	1	1.307	7.95	1
A	1	0.901	5.48	2.5
Experimental error	64	0.1644		

[a]A = aggregate type; B = aggregate gradation; C = asphalt viscosity; D = asphalt content; F = mixing temperature; and G = compaction temperature.

TABLE 4

ANALYSIS OF VARIANCE FOR MODULUS OF ELASTICITY AT 75 F

Source of Variation[a]	Degree of Freedom	Mean Squares	F-Value	Significance Level (percent)
BD	1	103.2	191.33	0.5
G	1	75.0	139.10	0.5
D	1	48.4	89.77	0.5
C	1	20.3	37.73	0.5
D_Q(quadratic)	1	20.3	37.61	0.5
BDG	1	18.8	34.91	0.5
B_Q(quadratic)	1	12.4	22.99	0.5
ABD	1	6.5	11.99	0.5
A	1	5.1	9.50	0.5
Experimental error	79	0.539		

[a]A = aggregate type; B = aggregate gradation; C = asphalt viscosity; D = asphalt content; F = mixing temperature; and G = compaction temperature.

One of the best methods of obtaining an overall view of the variation of a particular dependent variable or response created by changes in the levels of the significant main effect and interactions is to formulate the functional relationship that exists between a dependent variable and a number of independent variables. Unfortunately, this relationship is usually too complicated to be described in simple terms. If there is no prior knowledge of its form, the function is approximated by some simple polynominal function that contains the appropriate variables and is valid over some limited ranges of the variables involved. Such a mathematical function can be extremely valuable for predicting the values of dependent variables based on knowledge of the independent variables (9).

In this study an approximation of the functional relationship between the dependent and independent variables was obtained by combining in the form of a polynominal those main effects, quadratic effects, and interaction effects that were found in the analysis of variance to be of practical engineering significance.

A stepwise regression analysis was performed to develop equations that provided an acceptable estimate of the various dependent variables measured in the experiment. These equations can be used to make estimates of the different dependent variables within some standard error for combinations of the independent variables. Included with the equations are the standard error of estimate, $\hat{S}_r$, and the coefficient of determination, R^2. The terms A, B, C, D, and G are the levels of the various factors used in the experiment (Table 1).

It should be noted, however, that the predictive capabilities of the regression equations are valid only for the conditions and factors studied, i.e., those factors and levels given in Table 1. The use of any levels outside this factor space is not recommended.

The equation for tensile strength, psi, at 75 F is

$$\begin{aligned} S_T = {} & 150.8 - 5.027(B - 4.0) + 26.037(C - 9.1) - 12.691(D - 7.0) \\ & + 0.574(G - 250.0) - 10.929(B - 4.0)(D - 7.0) + 3.572(A - 1.0)(B - 4.0) \\ & (D - 7.0) - 0.0688(B - 4.0)(G - 250.0) - 0.0750(B - 4.0)(D - 7.0)(G - 250.0) \\ & - 3.2775(B - 4.0)^2 - 11.545(D - 7.0)^2 \end{aligned} \tag{1}$$

$\hat{S}_r = \pm 28$

$R^2 = 0.782$

The equation for total tensile strain at failure, micro-units, at 75 F is

$$\begin{aligned} \epsilon_T = {} & 1{,}372.9 + 96.28(A - 1.0) + 63.60(D - 7.0) - 3.147(G - 250.0) \\ & + 63.563(A - 1.0)(D - 7.0) \end{aligned} \tag{2}$$

$\hat{S}_r = \pm 380$

$R^2 = 0.310$

The equation for modulus of elasticity, 1×10^5 psi, at 75 F is

$$\begin{aligned} E = {} & 3.531 - 0.248(A - 1.0) + 0.6605(C - 9.1) - 0.3646(D - 7.0) \\ & + 0.01523(G - 250.0) - 0.2993(B - 4.0)(D - 7.0) + 0.07491(A - 1.0)(B - 4.0) \\ & (D - 7.0) - 0.002557(B - 4.0)(D - 7.0)(G - 250.0) - 0.09857(B - 4.0)^2 \\ & - 0.2570(D - 7.0)^2 \end{aligned} \tag{3}$$

$\hat{S}_r = \pm 0.853 \times 10^5$

$R^2 = 0.729$

The prediction capabilities of these 3 regression equations were established by several indicators and tests. Included among the indicators were multiple correlation coefficient, coefficient of determination, coefficient of variation, and standard error of estimate. The multiple correlation coefficient, which is generally denoted as R, is a measure of the linearity of the fit between the data and the regression equation, while the coefficient of determination R^2 indicates the portion of the total variation in the response variable that can be explained by the regression equation. The coefficient of variation is an indicator of the relative variation that can be expected. The standard error of estimate, $\hat{S}_r$, is the standard deviation of the errors of estimation. Normally, approximately two-thirds of the errors associated with the observed data will be less than the standard error of estimate, only about one-third of the errors will be more. In addition, under the same conditions, approximately 95 percent of the data will fall within a region bounded by 2 lines drawn parallel to the line of regression at a vertical distance of $\pm 1.96\ \hat{S}_r$.

One of the tests used to evaluate the regression equation was a test for lack of fit. The test essentially consisted of a significance test comparing the residual mean squares with the experimental error variance. The residuals for the regression equation contained all the available information on the failure of the fitted model to properly explain the observed variation in the response variable. If the model was correct; i.e., if the model fit the data, then the residuals contained only random variation that approximately equalled the experimental error variation. However, if the model was incorrect, the residuals contained systematic as well as random variations that were greater than the experimental error variation. The F-test for significance, then, indicated at some probability level ($\alpha = 0.01$ in this study) whether the regression equation properly explained the variation in the response variable.

The values of the indicators discussed as well as the results of the test for lack of fit are given in Table 5. Each regression equation was evaluated through the use of the indicators and a test for lack of fit. The parameters for tensile strength and modulus of elasticity (Table 5) indicate that the prediction capability of the regression equations was adequate. On the other hand, it is evident that the regression equation for total tensile strain was questionable; however, there was no significant lack of fit. The equation included the 3 factors of aggregate type (factor A), asphalt content (factor D), and compaction temperature (factor G). In this case a decision has to be made concerning the use of the equation. There are 2 alternatives. The first is to abandon the use of the equation and use the overall mean value of 1,370 micro-units as an estimate of the total tensile strain at failure. The basic argument for this approach is that because $R^2 = 0.310$ the equation explains only about 31 percent of the total variation in total tensile strain. The second alternative is to accept the equation with the reservation that there can be substantial variation in total tensile strain as evidenced by the relatively large standard error of estimate $\hat{S}_r = \pm 318$. The primary argument for this second approach is that a better approximation of total tensile strain than the mean can be obtained because there were 4 factors and their interactions that were found to be of practical engineering significance. On this basis, it is recommended that the regression equation be utilized.

TABLE 5

PARAMETERS FOR EVALUATING REGRESSION EQUATIONS

Response Variable	Correlation Coefficient	Coefficient of Determination	Coefficient of Variation (percent)	Standard Error of Estimate	Is There Significant Lack of Fit?
Tensile strength	0.8845	0.7823	14.2	±28.0	No
Total tensile strain	0.5564	0.3096	29.5	±318	No
Modulus of elasticity	0.8536	0.7286	20.8	$\pm 0.853 \times 10^5$	No

DISCUSSION OF RESULTS

Indirect Tensile Strength at 75 F

The variables included in the equation are aggregate type (factor A), aggregate gradation (factor B), asphalt viscosity (factor C), asphalt content (factor D), and compaction temperature (factor G). Table 6 gives estimated tensile strengths for different combinations of the 5 factors. Based on data given in this table, plots were developed indicating the relationship between asphalt content and compaction temperature for each aggregate at each of the 3 gradations. These are shown in Figure 3 for AC-5 asphalt cement. The effect of asphalt viscosity was linear; therefore, the tensile strengths for similar mixtures but with asphalts of different viscosities can be accounted for by adding the proper correction factors to the values obtained for AC-5 cement. For AC-10 and AC-20 asphalt cements, the correction factors are 13 and 31 psi respectively.

As shown in Figure 3, the specimens containing crushed limestone exhibited larger tensile strengths than specimens containing gravel. This behavior is attributed to the fact that the angularity, rough surface texture, and porosity of the limestone resulted in a better bond between the aggregate and the asphalt.

One of the striking aspects in all the relationships is the pronounced effect of compaction temperature on tensile strength; high compaction temperatures produced high

TABLE 6

ESTIMATED INDIRECT TENSILE STRENGTH OF AC-5 ASPHALT CEMENT AT 75 F

Compaction Temperature (deg F)	Asphalt Content (percent)	Crushed Limestone Aggregate (psi)			Gravel Aggregate (psi)		
		Fine	Medium	Coarse	Fine	Medium	Coarse
200	4.0	—	40.6	88.8	9.0	40.6	46.0
	4.5	2.3	66.0	103.5	38.0	66.0	67.8
	5.0	32.7	85.7	112.4	61.3	85.7	83.8
	5.5	57.3	99.5	115.5	78.7	99.5	94.1
	6.0	76.2	107.6	112.8	90.4	107.6	98.5
	6.5	89.2	109.9	104.4	96.4	109.9	97.2
	7.0	96.5	106.4	90.2	96.5	106.4	90.2
	7.5	98.0	97.2	70.2	90.9	97.2	77.3
	8.0	93.8	82.2	44.4	79.5	82.2	58.7
	8.5	83.8	61.4	12.9	62.3	61.4	34.3
	9.0	68.0	34.9	—	39.4	34.9	4.2
	9.5	46.4	2.5	—	10.7	2.6	—
	10.0	19.0	—	—	—	—	—
250	4.0	—	69.3	133.2	22.1	69.3	90.3
	4.5	19.2	94.7	144.1	54.9	94.7	108.4
	5.0	53.3	114.4	149.2	81.9	114.4	120.6
	5.5	81.7	128.2	148.6	103.1	128.2	127.1
	6.0	104.2	136.3	142.1	118.5	136.3	127.9
	6.5	121.1	138.6	130.6	128.2	138.6	122.8
	7.0	132.1	135.2	112.0	132.1	135.2	112.0
	7.5	137.4	125.9	88.3	130.2	125.9	95.4
	8.0	136.9	110.9	58.8	122.6	110.9	73.0
	8.5	130.6	90.2	23.5	109.2	90.2	44.9
	9.0	118.5	63.6	—	90.0	63.6	11.0
	9.5	100.7	31.3	—	65.0	31.3	—
	10.0	77.1	—	—	34.3	—	—
300	4.0	—	98.0	177.5	35.2	98.0	134.6
	4.5	36.0	123.4	184.7	71.7	123.4	148.9
	5.0	73.9	143.1	186.0	102.5	143.1	157.5
	5.5	106.0	156.9	181.6	127.4	156.9	160.2
	6.0	132.3	165.0	171.5	146.6	165.0	157.2
	6.5	152.9	167.3	155.5	160.1	167.3	148.4
	7.0	167.7	163.9	133.8	167.7	163.9	133.8
	7.5	176.7	154.6	106.3	169.6	154.6	113.5
	8.0	180.0	159.6	73.1	165.7	139.6	87.4
	8.5	177.4	118.9	34.1	156.0	118.9	55.5
	9.0	169.1	92.3	—	140.6	92.3	17.8
	9.5	155.1	60.0	—	119.4	60.0	—
	10.0	135.2	21.9	—	92.4	21.9	—

Note: Tensile strengths for mixtures with AC-10 and AC-20 asphalt cements can be obtained by adding 13 and 31 psi respectively to the values in this table.

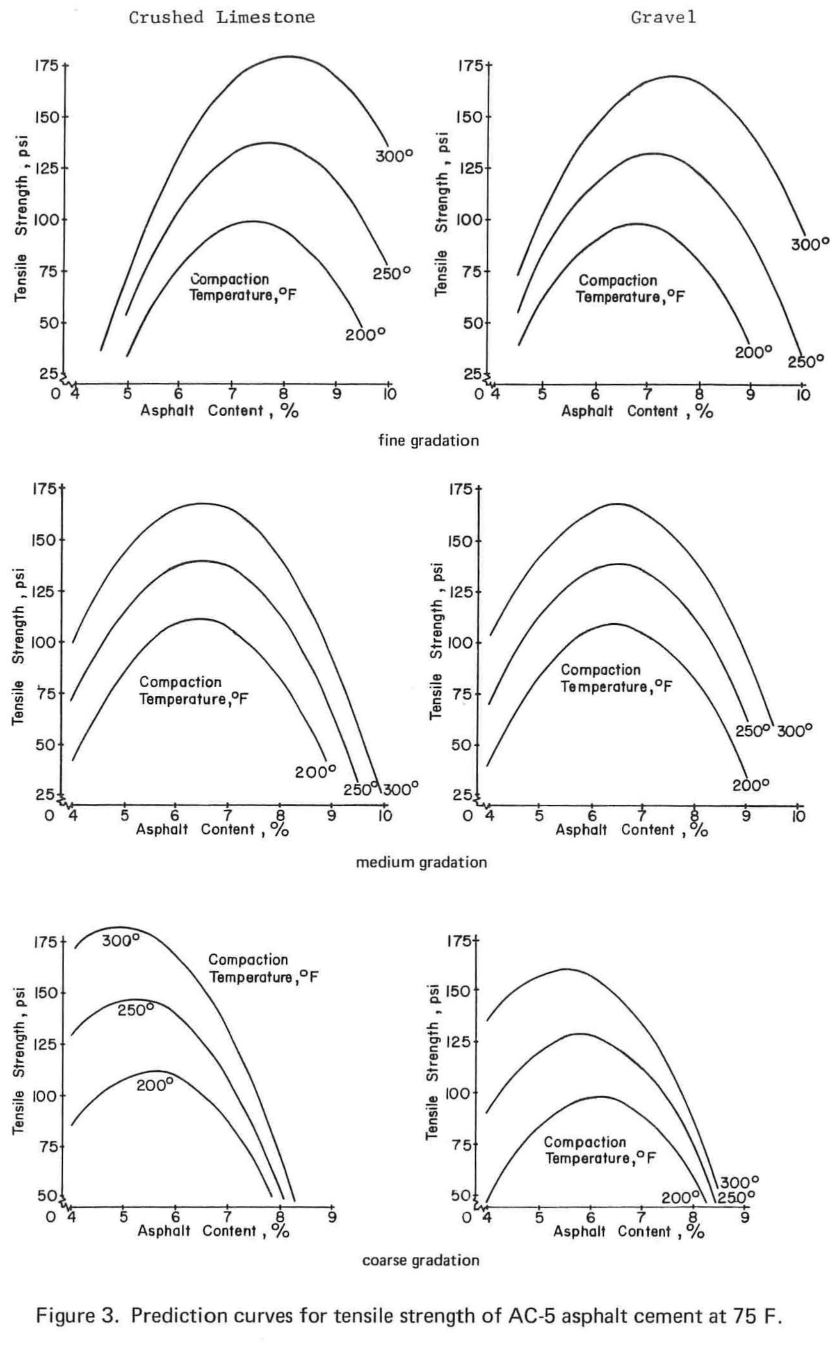

Figure 3. Prediction curves for tensile strength of AC-5 asphalt cement at 75 F.

tensile strengths. In addition, an optimum asphalt content occurred for each gradation of both aggregates; however, this optimum shifted slightly with increasing compaction temperatures for the fine and coarse gradations. For the fine gradations, the optimum asphalt content increased with increased compaction temperatures. On the other hand, for the coarse gradations, the optimum decreased with increased compaction; and for the medium gradation, the optimum asphalt content remained essentially constant. It

TABLE 7

ESTIMATED TOTAL TENSILE STRAIN AT FAILURE AT 75 F

Asphalt Content	Crushed Limestone (μ in.^{2})			Gravel (μ in.^{2})		
	200 F	250 F	300 F	200 F	250 F	300 F
4.0	1,435	1,275	1,120	1,245	1,090	930
4.5	1,435	1,275	1,120	1,310	1,150	995
5.0	1,435	1,275	1,120	1,370	1,215	1,060
5.5	1,435	1,275	1,120	1,435	1,280	1,120
6.0	1,435	1,275	1,120	1,500	1,340	1,185
6.5	1,435	1,275	1,120	1,565	1,405	1,250
7.0	1,435	1,275	1,120	1,625	1,470	1,310
7.5	1,435	1,275	1,120	1,690	1,535	1,375
8.0	1,435	1,275	1,120	1,755	1,595	1,440
8.5	1,435	1,275	1,120	1,815	1,660	1,505
9.0	1,435	1,275	1,120	1,880	1,725	1,565
9.5	1,435	1,275	1,120	1,945	1,785	1,630
10.0	1,435	1,275	1,120	2,010	1,850	1,695

can also be seen that the optimum asphalt content was higher for the specimens containing finer graded aggregates.

There are several factors involved in an explanation of the relationships shown in Figure 3. First, higher compaction temperatures produced greater fluidity of the asphalt cement, which allowed movement of the asphalt cement during compaction, thereby producing better distribution of the asphalt in the mixture and creating thinner films of asphalt connecting the aggregate particles.

The optimum asphalt contents for the finer graded mixtures were higher because more asphalt was required to cover the larger surface area associated with finer gradations. In addition, the increase in the optimum asphalt content with increased compaction temperatures was due to the fact that with increased fluidity during compaction the distribution of the asphalt was so improved that more of the fine particles could be bound together by asphalt films.

For the coarse graded mixtures, the optimum asphalt content decreased with increased compaction temperature because of the increased fluidity that allowed the aggregate particles to be adequately coated and connected with a smaller quantity of asphalt.

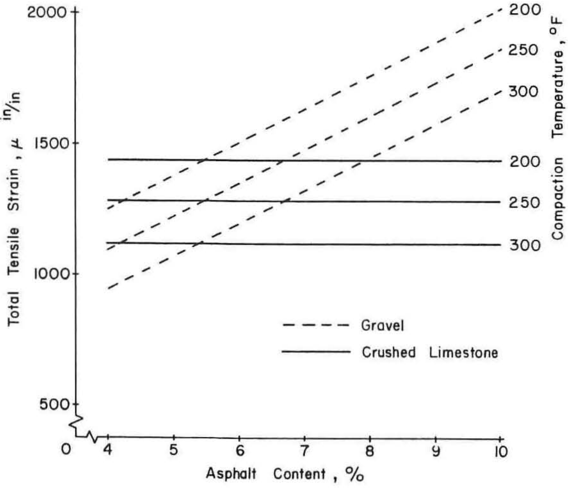

Figure 4. Prediction curves for total tensile strain at 75 F.

Total Tensile Strain at Failure at 75 F

Estimates of the total tensile strain based on the regression equations are given in Table 7, and a plot graphically illustrating the estimates is shown in Figure 4. Figure 4 shows that asphalt content had no effect on the total tensile strain of a mixture containing crushed limestone aggregate. On the other hand, for gravel-asphalt mixtures, the total tensile strain increased with increasing amounts of asphalt. For both aggregate types, the compaction temperature had a noticeable effect on total tensile strain; increased compaction temperature produced a decrease in the tensile strain at failure. This decrease in strain is attributed to the fact that the increased fluidity of the asphalt during compaction at the

higher temperatures resulted in improved distribution of the asphalt with thinner films connecting aggregate particles. Because deformation occurs primarily in the asphalt, these thinner films result in smaller strains.

The difference in the behavior of mixtures containing the 2 aggregate types is also attributed to the thickness of the asphalt films connecting the aggregate particles. The crushed limestone, which is highly porous, readily absorbed excess asphalt and thus tended to produce asphalt films of essentially the same thickness regardless of asphalt content. Thus, it would be expected that the failure strain would be essentially constant for all the asphalt-crushed limestone mixtures. The gravel, on the other hand, is relatively nonporous and does not tend to absorb the available asphalt. Therefore, as the amount of asphalt in the gravel mixture increased, the thickness of the asphalt films connecting the aggregate particles increased. The thicker asphalt films then allowed larger deformations to occur, resulting in larger strains.

Modulus of Elasticity at 75 F

A review of the modulus of elasticity equation shows that only 5 of the 7 variables were practically significant. These variables were aggregate type (factor A), aggregate gradation (factor B), asphalt viscosity (factor C), asphalt content (factor D), and

TABLE 8

ESTIMATED MODULUS OF ELASTICITY VALUES OF AC-5 ASPHALT CEMENT AT 75 F

Compaction Temperature (deg F)	Asphalt Content (percent)	Crushed Limestone Aggregate (1 x 10^5 psi)			Gravel Aggregate (1 x 10^5 psi)		
		Fine	Medium	Coarse	Fine	Medium	Coarse
200	4.0	—	1.401	2.485	—	0.906	1.091
	4.5	0.300	1.926	2.763	0.554	1.430	1.519
	5.0	0.942	2.322	2.913	1.046	1.826	1.818
	5.5	1.456	2.589	2.934	1.410	2.094	1.989
	6.0	1.841	2.728	2.827	1.645	2.233	2.032
	6.5	2.098	2.739	2.591	1.752	2.243	1.946
	7.0	2.226	2.621	2.226	1.731	2.125	1.731
	7.5	2.226	2.374	1.734	1.581	1.879	1.388
	8.0	2.098	1.999	1.112	1.302	1.504	0.916
	8.5	1.840	1.496	0.362	0.896	1.000	0.316
	9.0	1.455	0.864	—	0.360	0.368	—
	9.5	0.941	0.103	—	—	—	—
	10.0	0.298	—	—	—	—	—
250	4.0	—	2.163	4.014	—	1.667	2.619
	4.5	0.422	2.687	4.164	0.676	2.192	2.919
	5.0	1.192	3.083	4.186	1.296	2.588	3.091
	5.5	1.834	3.351	4.079	1.788	2.855	3.134
	6.0	2.347	3.490	3.844	2.151	2.994	3.049
	6.5	2.732	3.500	3.480	2.386	3.005	2.835
	7.0	2.988	3.382	2.988	2.492	2.887	2.492
	7.5	3.116	3.136	2.367	2.470	2.640	2.022
	8.0	3.115	2.761	1.618	2.320	2.265	1.422
	8.5	2.985	2.257	0.740	2.040	1.762	0.694
	9.0	2.728	1.625	—	1.633	1.180	—
	9.5	2.341	0.865	—	1.097	0.369	—
	10.0	1.826	—	—	0.432	—	--
300	4.0	—	2.924	5.542	—	2.429	4.148
	4.5	0.544	3.449	5.565	0.798	2.953	4.320
	5.0	1.442	3.845	5.459	1.546	3.349	4.364
	5.5	2.212	4.112	5.224	2.166	3.617	4.279
	6.0	2.853	4.251	4.861	2.657	3.756	4.066
	6.5	3.365	4.262	4.369	3.020	3.766	3.724
	7.0	3.749	4.144	3.749	3.254	3.648	3.254
	7.5	4.005	3.897	3.001	3.360	3.402	2.655
	8.0	4.132	3.522	2.124	3.337	3.027	1.928
	8.5	4.130	3.019	1.118	3.185	2.523	1.072
	9.0	4.000	2.387	—	2.906	1.891	0.088
	9.5	3.742	1.626	—	2.497	1.131	—
	10.0	3.355	0.737	—	1.961	0.242	—

Note: Modulus of elasticity estimates for mixtures with AC-10 and AC-20 asphalt cements can be obtained by adding 0.330 and 0.790 respectively to the values in this table.

compaction temperature (factor G). A number of moduli of elasticity for a variety of combinations of the 5 factors were estimated by utilizing the equation and are given in Table 8. In addition, plots indicating the relationship between asphalt content and compaction temperature for the 3 gradations and for each aggregate type are shown in Figure 5 for AC-5 asphalt cement. These relationships will change linearly with change in asphalt viscosity because the effect of asphalt viscosity was linear. Therefore, estimates of modulus of elasticity for mixes with AC-10 or AC-20 can be obtained by adding 0.330×10^5 and 0.790×10^5 respectively to the values obtained for AC-5.

The relationships between asphalt content and compaction temperature for the different gradations of crushed limestone and gravel are similar to those for tensile strength. In all the figures, the effect of compaction temperature is evident. In addition, optimum asphalt contents are evident and shift slightly with increased compaction temperature for the fine- and coarse-graded mixtures. The optimum asphalt content for fine-graded mixtures increased with increased compaction temperature, while the optimum for coarse-graded mixtures decreased with higher compaction temperatures. In addi-

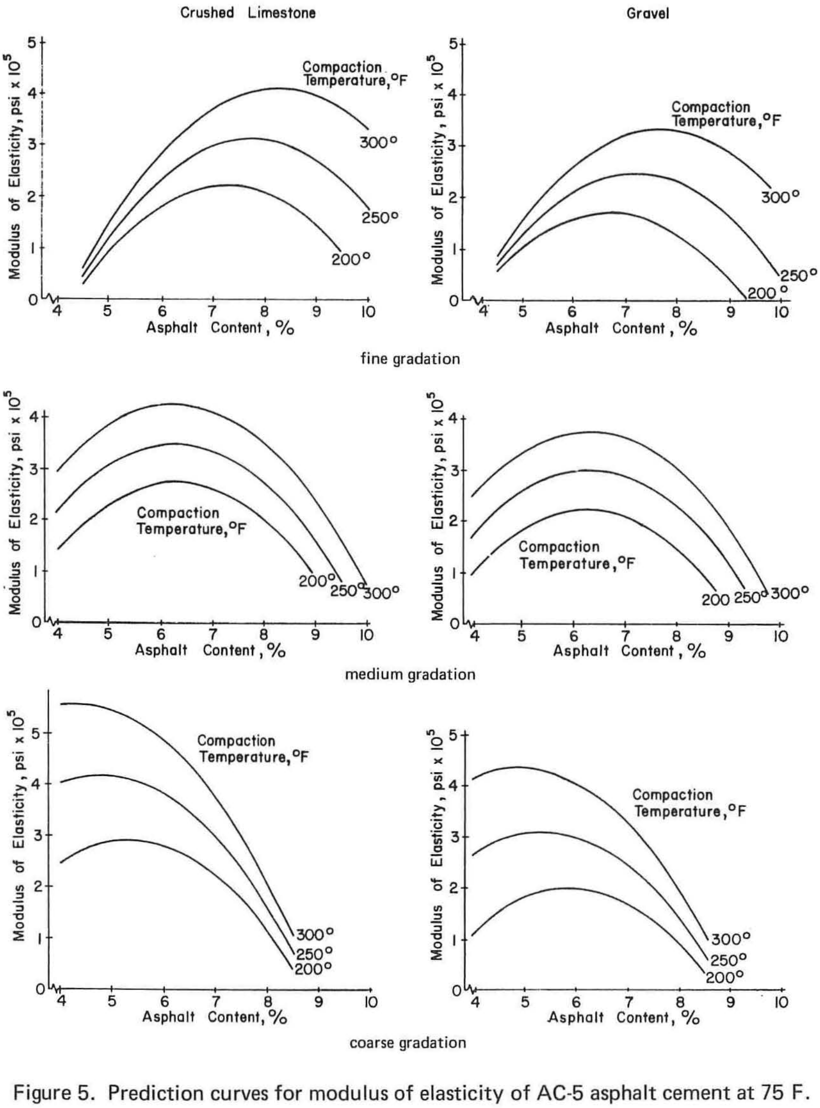

Figure 5. Prediction curves for modulus of elasticity of AC-5 asphalt cement at 75 F.

tion, it may be noted that specimens containing coarser graded aggregate exhibited larger modulus values.

Because there were similarities in the trends observed for modulus of elasticity (Fig. 5) and tensile strength (Fig. 3), the explanation of the relationship between tensile strength and the 5 significant (or important) variables can also be used to explain the relationship between modulus of elasticity and the same 5 variables.

There were, however, 2 distinct differences in the results for modulus of elasticity and tensile strength. First of all, although there were no differences in tensile strengths between asphalt-treated mixtures containing crushed limestone or gravel, there were differences in moduli of elasticity for the 2 different aggregate mixtures. Second, coarse-graded mixtures containing gravel generally exhibited higher moduli of elasticity but lower tensile strengths. Both are attributable to the difference in the failure strain behavior of mixtures containing the 2 aggregates.

Because modulus of elasticity is defined as the ratio of stress to strain, the modulus of elasticity from the indirect tensile test can be regarded as the ratio of tensile strength to tensile strain at failure. When this analogy is used, the modulus of elasticity of crushed limestone-asphalt mixtures should be related linearly to tensile strength because these mixtures exhibited constant tensile failure strains (Fig. 4). On the other hand, although the gravel-asphalt mixtures exhibited tensile strengths higher than those of the crushed limestone-asphalt mixtures, lower moduli of elasticity were produced because of greater tensile strains for the gravel-asphalt mixtures.

The second difference noted can be explained in a similar manner. The optimum asphalt content for the coarse-graded gravel-asphalt mixture was smaller than for the medium- and fine-graded gravel-asphalt mixtures; therefore, the tensile strains were lower and offset the lower tensile strengths, producing higher moduli of elasticity.

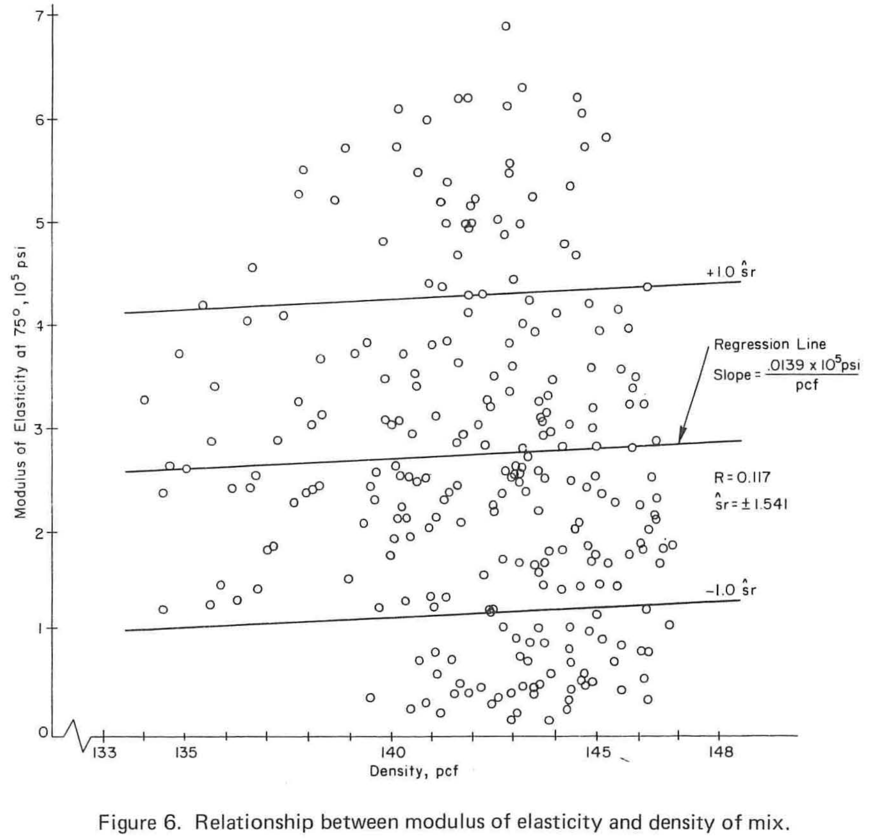

Figure 6. Relationship between modulus of elasticity and density of mix.

Correlation of Tensile Properties and Density

The density of an asphalt mix is of concern in design; however, it was difficult to control in this experiment because it was dependent on the factors involved in the mixing and compaction procedures. Thus, density was not an independent variable but was considered as a dependent or response variable similar to modulus of elasticity and tensile strength.

In general, it is considered that density is related to material properties, with higher densities corresponding to higher strengths. If this were true, then there should have been a good correlation between density and both modulus of elasticity and tensile strength.

In order to study this relationship, a correlation analysis was conducted that indicated that there was no trend or correlation between either tensile strength and density (Fig. 6) or modulus of elasticity and density (Fig. 7). The linear regression relationship relating the 2 tensile properties to density are included in the figure along with the correlation coefficient R and standard error of estimate $\hat{S}_r$. The slopes of the lines are very flat, indicating that the modulus of elasticity and strength were relatively independent of density.

Within the range of densities that occurred in this study, these results indicate that an increase in density may or may not be indicative of an increase in the modulus of elasticity or tensile strength. On the other hand, it has been shown that mixture variables such as aggregate type, gradation, asphalt viscosity, asphalt content, and compaction temperature can have a great influence on both the modulus of elasticity and tensile strength. Therefore, changes in density alone cannot be used as a measure of expected changes in tensile properties of the mix but must be accompanied by careful consideration of the factors involved in the mix design.

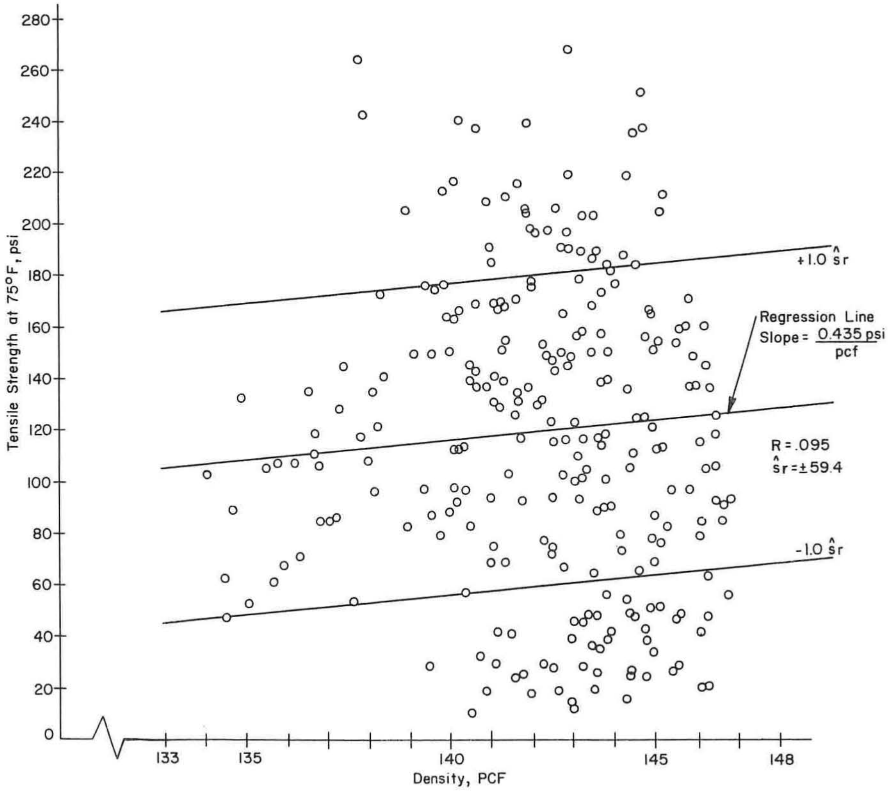

Figure 7. Relationship between tensile strength and density of mix.

CONCLUSIONS AND RECOMMENDATIONS

This paper presents equations for estimating the modulus of elasticity, tensile strength, and total tensile strain at failure for a variety of asphalt-treated materials at 75 F. The equations for modulus of elasticity and tensile strength are shown to be very reliable and can be used to predict the variations in these 2 responses with changes in aggregate type, gradation, asphalt viscosity, asphalt content, and compaction temperature. The equation for total tensile strain is not as reliable as the other two, but it can be used to provide better estimates of tensile strain for these materials than those currently available. In all cases, the decision to use these equations must be based on the error that can be tolerated. Nevertheless, it is felt that these equations are the best estimators currently available.

Estimates of modulus of elasticity, tensile strength, and total tensile strain at failure for a variety of combinations of the independent variables are given in Tables 6 through 8. Graphical representations of the interrelationships among the response variables, modulus of elasticity, tensile strength, and tensile strain, and a number of the mix variables are shown in Figures 3, 4, and 5 respectively and indicate the dominant effect of compaction temperature on all 3 tensile properties.

A number of variables must be considered when the tensile properties of asphalt-treated materials are evaluated. When the modulus of elasticity and tensile strength of such mixtures are evaluated, consideration must be given to aggregate type, gradation, asphalt viscosity, asphalt content, and compaction temperature. Aggregate type, gradation, and compaction temperature must also be considered when total tensile strain at failure is evaluated.

Because optimum asphalt contents were detected for the modulus of elasticity and tensile strength, it would appear that the indirect tensile test can be used to obtain optimum mix designs. The optimum asphalt content obtained, however, will depend on the aggregate type, aggregate gradation, and compaction temperature. Although a set of design tests may be needed to complement the results, the tables presented can be used to provide preliminary estimates and narrow the range of investigation in the laboratory.

Because there was no correlation between either modulus of elasticity and density or tensile strength and density for the conditions of the test, changes in density alone cannot be used as a measure of expected changes in tensile properties of the mix but must be accompanied by careful consideration of the factors involved in the mix design.

The effect of compaction temperature could explain some of the differences observed in the past between laboratory and field results because most laboratory procedures involve preparation of materials at certain fixed compaction temperatures. If the mixtures are compacted in the field at temperatures much different from those used in laboratory tests, then certainly, as evidenced by the results of the study, the mixture cannot be expected to perform in the field as predicted in the laboratory. Closer control of compaction temperature in the field through specification requirements could produce mixture properties closer to those design mixtures established in the laboratory and could substantially increase uniformity of mixtures along the length of the highway.

Present laboratory test procedures should be extended to include the evaluation of effects of changes in compaction temperature.

ACKNOWLEDGMENTS

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INFLUENCE OF PROCESSING PROCEDURES ON STRENGTH OF SAND STABILIZED WITH CATIONIC BITUMEN EMULSION

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This is the first of 2 papers that describe a laboratory study of the factors affecting the shear strength of a uniformly graded sand stabilized with cationic bitumen emulsion. Triaxial tests have been used to ascertain the effects of emulsion content on the cohesion and angle of shearing resistance of stabilized sand. Unconfined compression tests were used to investigate the influence of the viscosity of the base bitumen and also the effect of varying the initial moisture content of the sand. The addition of fillers of various types such as cement, hydrated lime, crushed limestone, and fine sand has been experimented with, and their influence on shear strength has been determined. A study of the effect (on strength) of changing the order of addition of ingredients during mixing is described. The effect of aging on the unconfined compressive strength of stabilized sand is reported.

•IN MANY AREAS of the world, particularly in underdeveloped countries, there is a shortage of natural materials suitable for road construction. In such locations, some form of in situ soil stabilization can often provide an alternative means of construction especially suitable where the intensity of traffic is low. Cement is the most commonly used stabilizing agent; but in the case of fine uniformly graded and rounded sand, often found in arid or semi-arid areas, cement stabilization has not been a particularly successful solution.

Some success has been achieved in stabilizing sands with bituminous binders. A useful bibliography on bitumen stabilization has been published by the Road Research Laboratory (1). This paper is concerned with the use of cationic emulsion for stabilizing medium to finely graded sand, which normally would be difficult to treat.

Two main categories of emulsion exist. The first is the more commonly used "anionic" emulsion, the development of which took place in the 1930's. The other, more recently developed, is "cationic" emulsion, which was first prepared in Europe and has been used on a commercial scale in the United States since 1958 (2, 3). An investigation of the effects of the variables of manufacture on its properties was carried out by Sauterey et al. (4); information on cationic emulsifiers has been published by Armour Hess Chemicals, Ltd. (5).

There was a paucity of information on the properties of sand-cationic emulsion mixes, and the authors considered that it would be useful to carry out a laboratory study of these so that the information would be available to assist highway engineers concerned with mix design, field processing, and curing. Full-scale field tests will doubtless be necessary before reliable specifications can be written, but at least the number of areas of ignorance has been reduced. Two papers will describe the results of this work. This is the first and is concerned with the effects of mixing proportions, mixing process, compaction, and curing on the strength of sand-cationic emulsion mixes.

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Siliceous aggregates have negatively charged surfaces; therefore, cationic emulsions adhere to the great majority of construction aggregates much more readily than anionic emulsions do. This means that bitumen deposited on aggregate surfaces will tend to adhere and will resist being stripped off in the presence of water. This superior adhesion property is the chief advantage of using cationic rather than anionic emulsion and accounts for the superior rewet strength and water absorption properties of soils stabilized with cationic emulsions. Also, because of their affinity to aggregate surfaces, cationic emulsions unlike anionic do not break primarily because of a loss of water and, therefore, do not depend on the slow evaporation of water. Consequently, on breaking, some strength is developed immediately, although this may not be very great. They can, therefore, be used on wet and cool aggregates as well as on dry warm surfaces.

One disadvantage of cationic emulsion compared with anionic is that the mixing stability tends to be low, i.e., the emulsion "breaks" after only a brief period of mixing. This breaking time may be controlled to some extent by selecting various proportions and types of emulsifier; but if the correct selection of these is not made, then inadequate coating will result and the ultimate strength of the mix will be impaired.

CATIONIC EMULSIONS USED

The viscosity of an emulsion starts to rise rapidly when its bitumen content is increased by more than 60 percent (5); hence, all the emulsions used in the investigation had bitumen contents within the range 56 to 64 percent.

Some emulsions were made up with Shell Mexphalt bitumen having a viscosity grade of 190 to 210 penetration and with a cationic emulsifier known as Redicote E11 (6), abbreviated RE11. Initial aggregate coating tests proved that 0.4 percent emulsifier was required to make the emulsion stable enough for adequate coating during mixing. Other emulsions using bitumen with viscosities 90 to 110 and 60 to 70 penetration required 0.6 and 0.75 percent respectively of emulsifier to ensure adequate mixing stability. Generally, mixing stability was improved by keeping the pH values only slightly acidic.

Two other types of emulsion were used. A 90 to 110 penetration bitumen was emulsified with 1.09 percent Duomeen Tallow (5), abbreviated DT and supplied by Armour Hess Chemicals, Ltd. The other type of emulsion, referred to as Lion, was prepared by Lion Emulsions, Ltd. Two such emulsions were used, one having 60 to 70 penetration and the other 40 to 50 penetration bitumen. The properties of these emulsions are given in Table 1.

PROPERTIES OF SAND CHOSEN FOR INVESTIGATION

It was desired to carry out the study by using a sand similar to many naturally occurring sands, which would be difficult to stabilize. Leighton Buzzard sand having the fairly uniform grading shown in Figure 1 was used. Its properties are given in Table 2.

TABLE 1

PROPERTIES OF EMULSIONS USED

Emulsifier		Viscosity (penetration)	Emulsion Viscosity (deg Engler)	Bitumen Content (percent)			pH	Typical Residue on No. 100 Sieve (percent)
Type	Percent			Specified	Measured	At Top of Container After 9 Months		
E11	0.4	190 to 210	15	64	61	1.5	5.7	0.25
	0.4	90 to 110	15	64	60	2.5	5.7	0.25
	0.6	90 to 110	15	64	60	5	5.7	0.25
	0.75	60 to 70	15	58	57	–	4.9	0.25
DT[a]	1.09	90 to 110	15	64	61	58.3	–	0.25
Lion	–	60 to 70	5.4	56	55	39	–	0.05
	–	40 to 50	5.4	56	55	–	–	0.05

[a]Formulation in parts by weight of emulsion was 0.13 percent Duomeen T, 0.13 percent Arquad T50, 0.13 percent Ethoduomeen T/13, and 0.7 percent conc. HCl.

EFFECT OF EMULSION CONTENT ON STRENGTH OF STABILIZED SAND

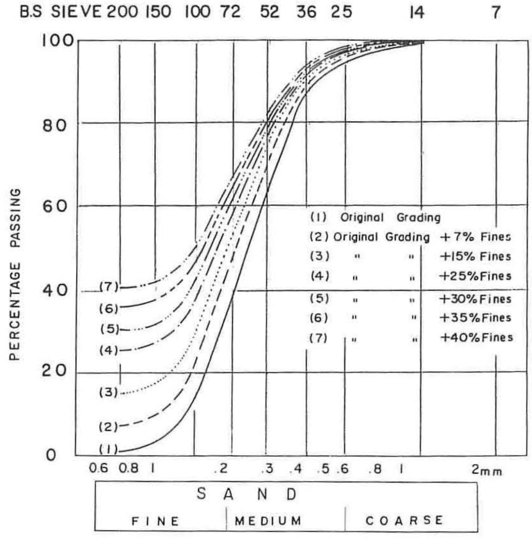

Figure 1. Grading of sand used in the investigation.

Because the shear strength of sand is due entirely to its frictional properties, its strength is a function of its porosity and the confining stresses to which it is subjected. The addition of bitumen as a binder has the effect of giving unconfined sand some cohesive strength; but, in general, it also reduces the angle of shearing resistance. Different proportions of bitumen in sand will result in varying values of cohesion and angles of shearing resistance, but it is important to realize that the strength exhibited by stabilized sand may also be influenced by the numerous variables, both natural and man-made (Fig. 2).

For a given sand, stabilized with a particular percentage weight of emulsion, the strength actually measured can vary significantly with type of test used to measure strength; rate of loading; efficiency of mixing; density to which the material is compacted; moisture content; confining stresses to which the specimen is subjected; temperature at which it is mixed, cured, and tested; and age of the specimen.

Because mixes containing different emulsion contents have their strength changed by different amounts by these variables, the optimum emulsion content for one set of conditions is generally different from that for another set of conditions. This is also true of other bituminous-bound materials (8, 9). Hence, it is not possible to think in terms of there being an "optimum" emulsion content to give maximum shear strength, unless most of these conditions are also specified. Further, the emulsion content that gives maximum shear strength under a defined set of conditions will, in general, not be the content at which the highest dry density can be achieved by a given amount of compactive effort. Nor will the specimen having the highest strength necessarily have the greatest stiffness. It is also true that the mix that gives the highest strength may not be the least permeable or may not have the greatest resistance to absorption to water.

TABLE 2

PROPERTIES OF LEIGHTON BUZZARD SAND USED IN STUDY

Property	Amount
Chemical	
Silica, percent	98.81
Alumina, percent	0.35
Other metal oxides, percent	0.54
Ignition loss, percent	0.20
Total	99.90
Physical	
Maximum dry density (B.S. 1377), pcf	101.5
Optimum moisture content (B.S. 1377), percent	10.5
Surface area, cm^2/gram	119.5
Specific gravity	2.65
Angularity (7)	1.1
Sphericity	0.82
Minimum porosity (at 0 percent moisture content), percent	31.3
Uniformity coefficient	1.95
Internal porosity, percent	0.4 to 1.4

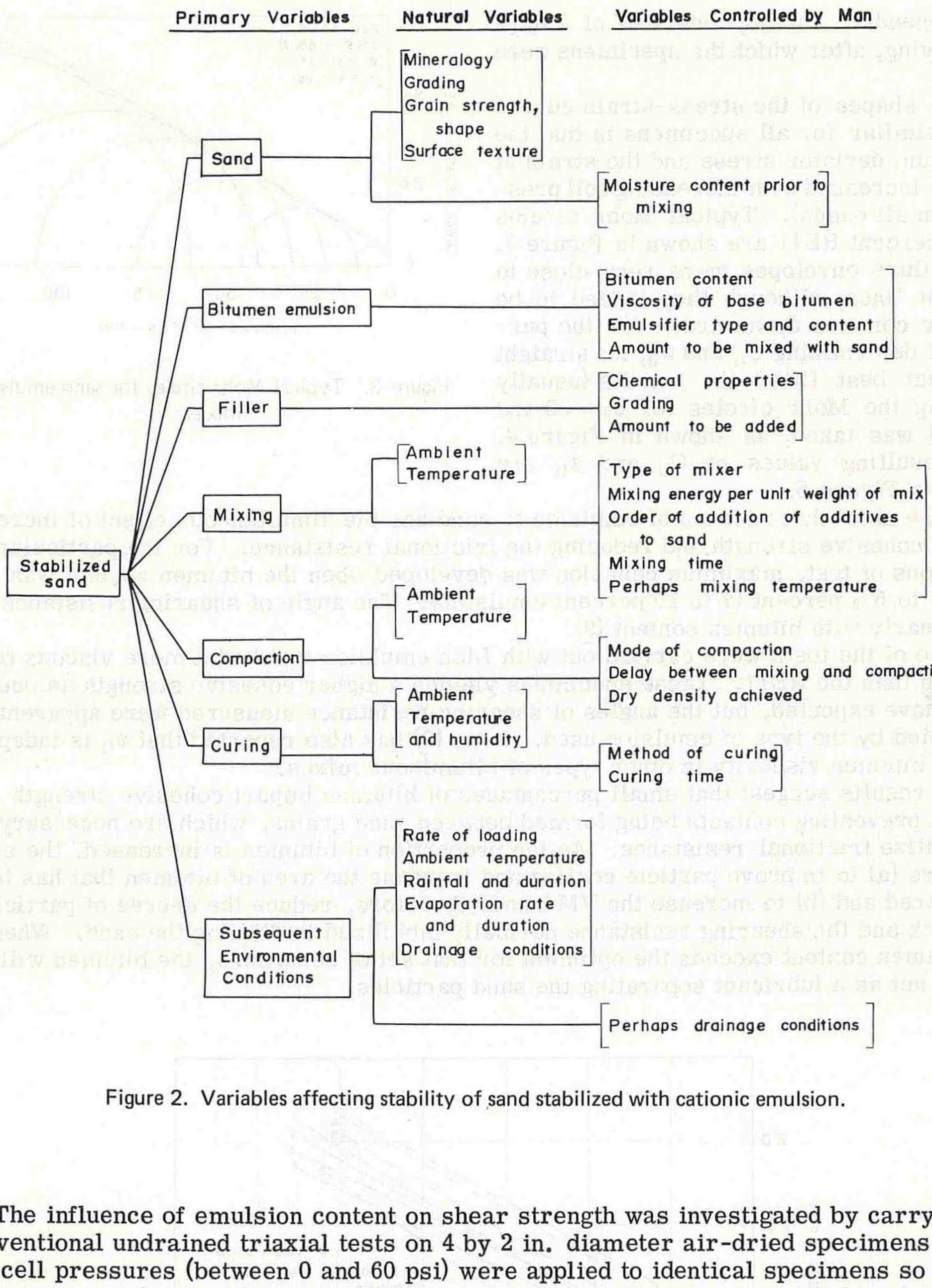

Figure 2. Variables affecting stability of sand stabilized with cationic emulsion.

The influence of emulsion content on shear strength was investigated by carrying out conventional undrained triaxial tests on 4 by 2 in. diameter air-dried specimens. Varying cell pressures (between 0 and 60 psi) were applied to identical specimens so that the apparent cohesion, C_u, and the angle of shearing resistance, ϕ_u, could be determined. All specimens were deformed at a constant rate of strain of 1.1 percent per minute under constant temperature conditions (18 C).

Several batches containing different emulsion and water contents were mixed. Two types of emulsion were used: RE11 with 60 to 70 penetration and Lion with 40 to 50 penetration. Specimens containing 3, 6, 9, and 13 percent RE11 emulsion with respectively 5, 4, 3.5, and 3 percent additional water were made. Other specimens containing 3, 5, and 7 percent Lion emulsion with respectively 3, 3, and 2 percent additional water were also made. The proportions of additional water used approximated those found from previous experiments to give the highest unconfined compressive strengths. All specimens were given standard mixing and compaction as described in

the Appendix. Curing consisted of 7 days air-drying, after which the specimens were tested.

The shapes of the stress-strain curves were similar for all specimens in that the maximum deviator stress and the strain at failure increased with increasing cell pressure (in all cases). Typical Mohr circles for 3 percent RE11 are shown in Figure 3. The failure envelopes were very close to straight lines, although they tended to be slightly concave downward. For the purpose of determining C_u and ϕ_u, the straight line that best fitted the results (usually touching the Mohr circles for $\sigma_3 = 20$ and 40 psi) was taken, as shown in Figure 4. The resulting values of C_u and ϕ_u are shown in Figure 5.

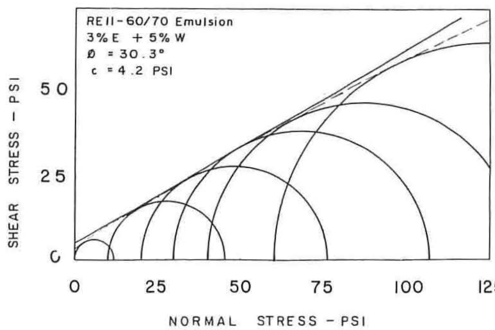

Figure 3. Typical Mohr circles for sand-emulsion mixes.

These show that addition of emulsion to sand has the simultaneous effect of increasing the cohesive strength and reducing the frictional resistance. For the particular conditions of test, maximum cohesion was developed when the bitumen content was about 4 to 5½ percent (7 to 10 percent emulsion). The angle of shearing resistance fell linearly with bitumen content (9).

Some of the tests were carried out with Lion emulsion that had a more viscous base bitumen than the RE11. These specimens yielded a higher cohesive strength as one might have expected, but the angles of shearing resistance measured were apparently unaffected by the type of emulsion used. Lees (9) has also reported that ϕ_u is independent of bitumen viscosity in other types of bituminous mixes.

The results suggest that small percentages of bitumen impart cohesive strength without preventing contacts being formed between sand grains, which are necessary to mobilize frictional resistance. As the proportion of bitumen is increased, the effects are (a) to improve particle coating and increase the area of bitumen that has to be sheared and (b) to increase the VMA and, therefore, reduce the degree of particle interlock and the shearing resistance normally mobilized in dilating the sand. When the bitumen content exceeds the optimum for that set of conditions, the bitumen will tend to act as a lubricant separating the sand particles.

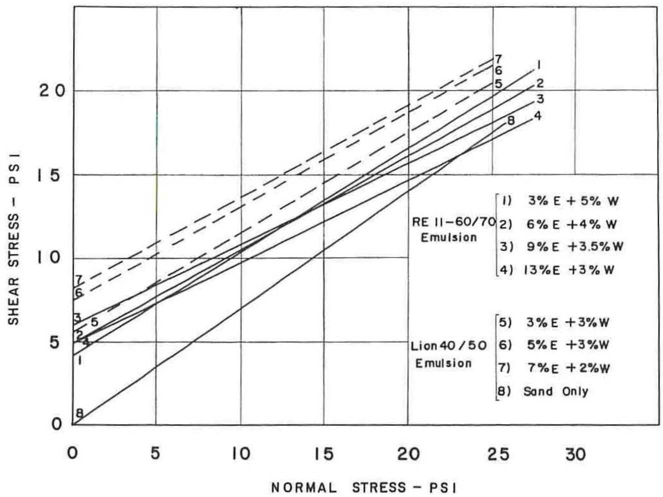

Figure 4. Mohr envelopes for various emulsion contents.

EFFECT OF INITIAL MOISTURE CONTENT OF SAND

From the literature, it can be concluded that the presence of moisture in a soil prior to the addition of emulsion aids the processing of sand bitumen mixes generally in the following ways: It facilitates mixing and distribution of binder; it provides sufficient liquid in the mix to bring it closer to optimum for compaction; and it hydrates any chemically active filler that may be present.

While the presence of moisture may delay the breaking of stable anionic emulsion and the adherence of the bitumen to the surfaces of sand particles, cationic emulsions are not so affected, owing to the preferential absorption of bitumen to negatively charged surfaces.

The effect of the initial moisture content on the unconfined compression strength of sand emulsion mixes was investigated by testing specimens in sets of three, each set being made up with sand having a different initial moisture content. Results were obtained by using emulsion contents of 3 and 5 percent (Lion with 40 to 50 penetration bitumen). All samples were given the same standard vibration compaction and cured for 7 days in air-dry conditions. The results of these tests, shown in Figure 6, indicated that there was an optimum initial moisture content (3 percent) that gave the highest unconfined compressive strength.

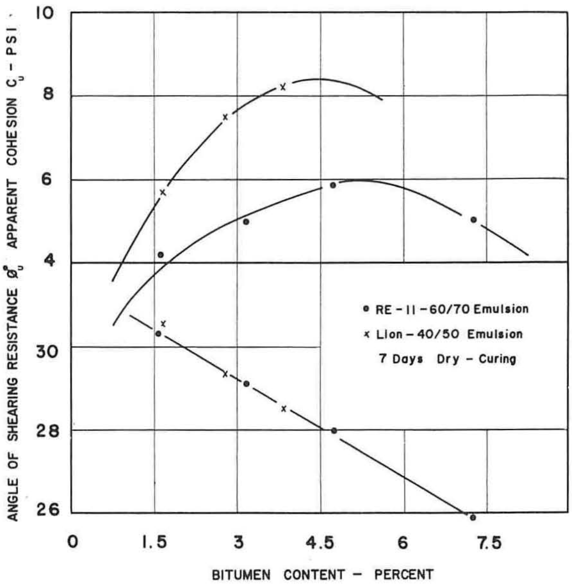

Figure 5. Effect of bitumen content on cohesion and angle of shearing resistance.

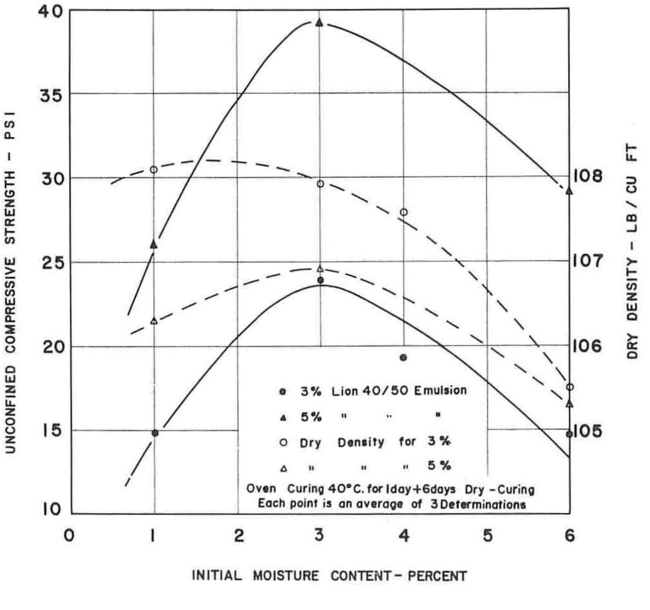

Figure 6. Effect of initial moisture content on strength.

For both emulsion contents, the dry density achieved by compaction was only marginally affected by increasing the initial moisture content from 1 to 3 percent, but the compressive strength increased significantly presumably because of improved distribution of bitumen and coating of the particles. However, it may be seen that, when the initial moisture content was more than 3 percent, the dry density achieved was deleteriously affected and there was a concomitant reduction in strength. It was noted that specimens having an initial moisture content of more than 3 percent became very "spongy" and soft during compaction.

The results of these tests prove that as much water should be present as will not deleteriously affect efficient compaction. If sand is dry, water should either be added to the sand prior to mixing it with emulsion or be added to the emulsion, thus decreasing its viscosity and increasing its mixing stability.

EFFECT OF EMULSION TYPE AND BITUMEN VISCOSITY

A perfunctory investigation was carried out to observe the influence of the viscosity of the emulsion base bitumen on the strength of the stabilized sand and to ascertain whether the type of emulsion affected strength.

Specimens were made up for unconfined compression tests by using varying proportions of different emulsions. The standard mixing and compaction procedure was used, but curing consisted of storing specimens for 7 days in a sealed condition at laboratory temperature.

The results of these tests, shown in Figure 7, reflect the very low strengths typical of specimens that have not been allowed to dry out during curing. It is clear that the emulsions having the lowest base bitumen penetrations or highest bitumen viscosity gave the highest strength. The optimum emulsion content that yielded the highest strength in each case was not the same. The reason for this was not investigated, but differences in emulsion properties such as the stability and the emulsion viscosity suggest that they are responsible.

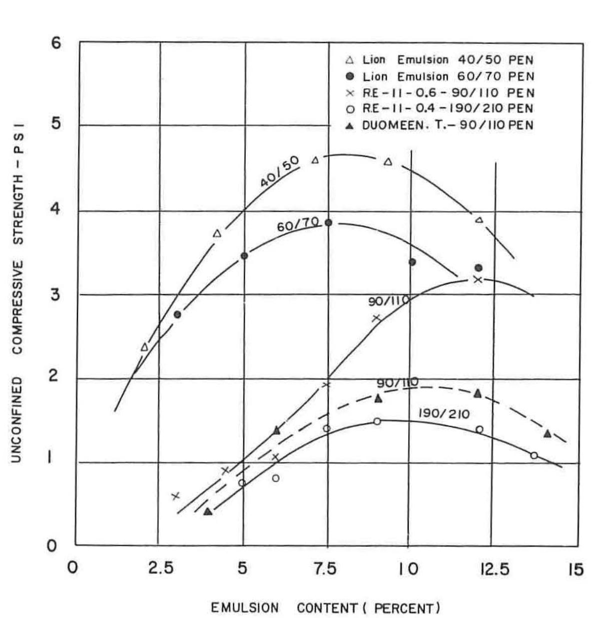

Figure 7. Effect of type of emulsion and viscosity of base bitumen on strength.

EFFECT OF ADDITION OF FILLER

The sand used for the experiments described was uniformly graded and had a relatively low stability compared with materials containing a wider range of particle sizes. It is well known that the addition of fines to a uniformly graded material will have the effect of increasing its mechanical stability. The increase in strength is accentuated if the fines are pozzolanic. This fact is well shown by the results of unconfined compression tests carried out on sand stabilized by addition of filler only. Figure 8 shows how the strength was significantly increased by addition of either cement, hydrated lime, or crushed limestone passing the No. 200 B.S. sieve. The results shown were for specimens mixed with 9 percent water, vibration-compacted to the same density (within practical limits), and wet-cured for 7 days before being tested.

Other sand-filler-emulsion mixes having 9 and 12 percent DT emulsion with 5 and 10 percent hydrated lime and 7 and 12.5 percent crushed limestone dust were made. Standard compaction was applied, and the specimens were tested after 7 days wet-curing. The results of these tests are shown in Figure 9.

As one might have expected from the results shown in Figure 8, mixes containing the higher proportions of filler gave the highest strength. By comparing Figure 8 with Figure 9, it is evident that for samples containing a high filler content the emulsion contributed nothing to the strength. For those containing low amounts of filler, the emulsion increased the strength significantly. Figure 9 might lead one to suppose that there is an optimum emulsion content of about 9 percent giving highest strength, but this is unlikely to be true for specimens of a different age or for specimens cured in a dry instead of a wet condition.

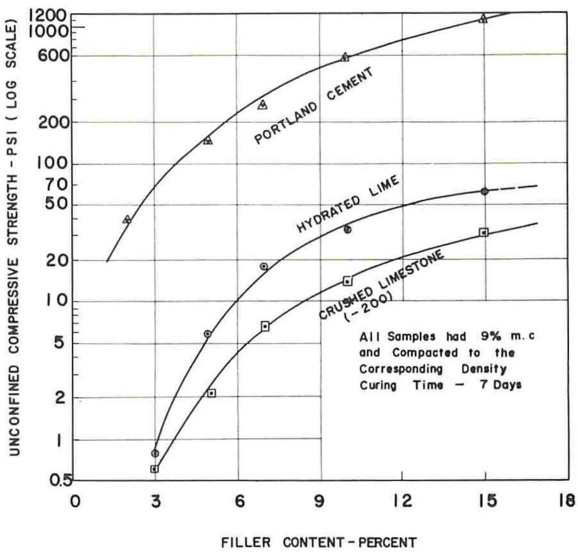

Figure 8. Effect of filler type and content on unconfined compressive strength of sand.

EFFECT OF GRADING OF SAND

A comprehensive study of the effects of grading would alone require a long-term investigation. However, by varying the content of sand passing a No. 200 B.S. sieve, it was possible to study the grading variable that probably has the major influence on engineering properties.

A number of samples of sand were made up to the various gradings shown in Figure 1 by adding various percentages of fine Leighton Buzzard sand passing through a No. 200 sieve. These various samples were stabilized with both 9 and 12 percent DT emulsion, the specimens being prepared, cured, and tested in exactly the same way as described earlier.

Addition of fines had an effect similar to that of the addition of fillers in that it increased strength as shown in Figure 10. However, the strengths resulting from addition of fines were appreciably lower than those achieved by fillers, presumably because of the chemically less active nature of the fines.

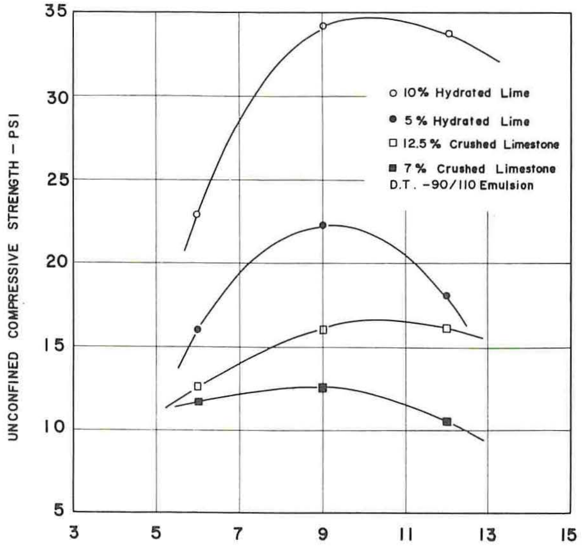

Figure 9. Effect of filler and emulsion on strength.

EFFECT OF CEMENT AND EMULSION ON COMPACTION

Compaction tests were carried out in turn on (a) sand with water, (b) sand plus 10 percent cement

with water, (c) sand plus 10 percent cement with various emulsion contents, (d) sand with various emulsion contents, and (e) sand plus various emulsion contents with water.

All materials were mixed in a ½-cu ft paddle mixer. The ingredients were added to the mixer in the following order: sand, distilled water, cement filler, and emulsion. (Experiments on mixing procedure, which were run subsequently, showed that filler should have been added after the emulsion in order to give the highest strength.)

The Kango hammer described in the Appendix fitted with a 4-in. nominal diameter piston was used to compact the material into a standard AASHO mold. Two minutes of vibration were given to each of the 3 layers.

The results of these tests are shown in Figures 11 and 12. Figure 11 shows the typical compaction curve for sand with its double optima at 0 and 9 percent moisture contents. The addition of cement only to the sand had the effect of increasing the dry density by an almost constant amount over the range of moisture contents tested.

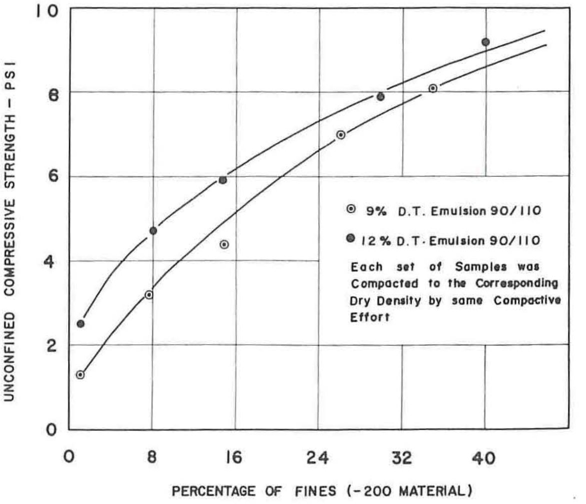

Figure 10. Effect of fines on compressive strength of emulsion stabilized sand.

Emulsion had the effect of reducing the dry density, and this fell with further increases in the emulsion content so that there did not appear to be an optimum emulsion content to give maximum dry density. Addition of cement as well as emulsion again increased the density, as one might have expected.

The tests in which both emulsion (RE11 with 60 to 70 penetration) and water contents were varied showed that the addition of emulsion had the effect of decreasing the optimum water content but increasing the optimum liquid content (Fig. 12). Addition of emulsion, however, had the effect of reducing the dry density so that the percentage of air voids at the maximum dry density obtained for each emulsion content only decreased slightly. For practical purposes, it could be assumed to remain constant at about 17 percent air voids.

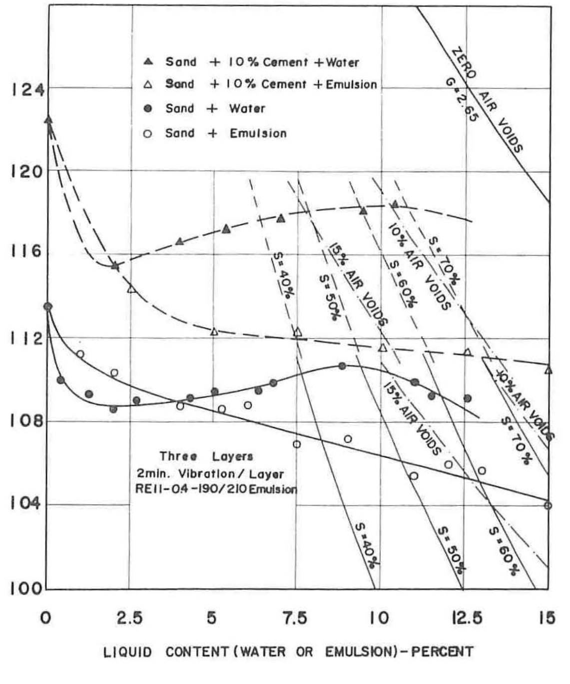

Figure 11. Results of vibrational compaction tests on sand and stabilized sand.

EFFECT ON STRENGTH AND STIFFNESS

Two series of tests were completed: The first examined the ef-

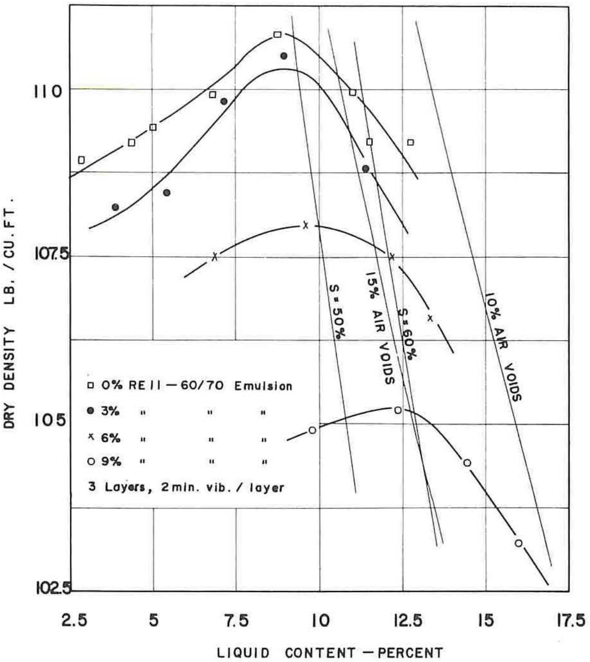

Figure 12. Effect of liquid content on the vibration compaction characteristics of sand-emulsion mixes.

fects of emulsion content with and without 10 percent cement filler added, while the second studied the effect of adding 10 percent cement and emulsion to sand, having varying initial moisture contents. All specimens were compacted by the standard equipment described in the Appendix, but compaction was adjusted so that specimens would have dry densities corresponding to the liquid and cement contents shown in Figure 11. Where the moisture content was adjusted, the dry densities to which materials were compacted were interpolated from the density curve for sand plus cement plus water and sand plus cement plus emulsion shown in Figure 11. All specimens were wet-cured before being tested in unconfined compression.

Figure 13 shows the results of the first series of tests. The effect of addition of cement is obvious, and it is clear that the cement was acting much more as a stabilizing agent than as a filler. The sand containing emulsion without a cement filler had a strength of only a few pounds per square inch.

Figure 14 shows that, for different emulsion contents, different quantities of water had to be added to give maximum strength, and the optimum liquid content increased with increase in emulsion content.

It was evident that the presence of emulsion only contributed to the strength of the sand-cement mix when the moisture content was low and that emulsion added to mixtures having adequate water for hydration of the cement had a marked detrimental effect on strength. Addition of emulsion also resulted in a concomitant reduction in stiffness as measured by the slopes of the stress-strain graphs obtained.

From this, it may at first sight be concluded that, if it is economical to use cement as a filler, it would be better to stabilize the sand with cement rather than to add emulsion. This would be true if unconfined compressive strength were the only criterion for satisfactory stabilization. However, a sand-cement mix is very stiff and, when laid on a relatively flexible sand subgrade, may well fail in tension when loaded. According to Burmister's analysis of a 2-layer elastic system, if the top layer is very

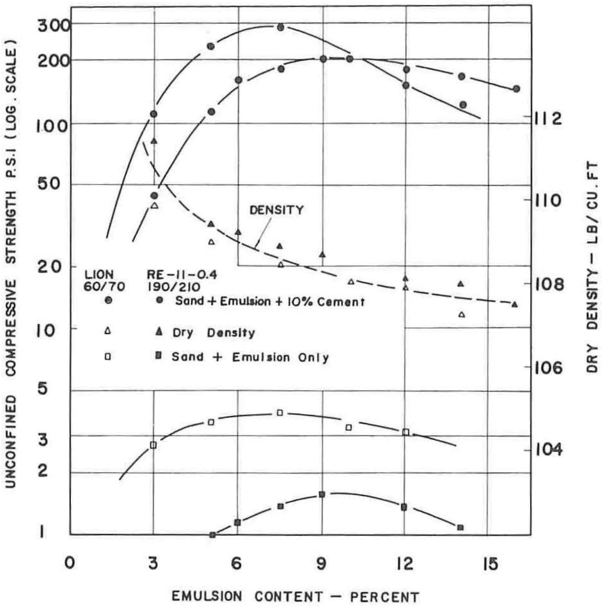

Figure 13. Effect of emulsion content on the compaction and strength of stabilized sand and sand-cement.

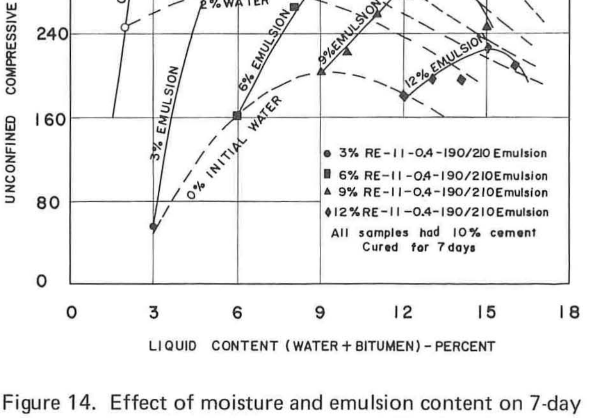

Figure 14. Effect of moisture and emulsion content on 7-day strength of cement-emulsion stabilized sand.

stiff relative to the subgrade, it is subjected to high tensile stresses on the interface between the 2 layers. If the stiffness of the top layer is reduced, the effect is to reduce the tensile stress in the material at the expense of a small increase of shear stress in the subgrade. Although the stiffness or modulus of deformation of a cement-stabilized sand may be of the order of 10^5 to 5×10^5 psi, the modulus of bitumen-stabilized sand is of a lower order around 10^4 to 10^5 psi, depending on the density, bitumen, and water content. Hence, although soil cement has a high compressive strength, it may fail in tension while a more flexible material with a lower compressive strength may well survive under the same loading conditions.

EFFECT OF MIXING PROCEDURES

A study was made of the effect on the 7-day compressive strength first of the mixing time and second of the order of addition of ingredients during mixing. In both cases Lion emulsion with 60 to 70 penetration was used. In the first study, 3 different liquid contents were selected: 6 and 9 percent emulsion with no water and 6 percent with 3 percent water. All batches (having a constant weight of sand) contained 2 percent cement that was first mixed dry with the sand before addition of emulsion and water. The batches were all mixed in a food mixer set at constant speed. The effect of mixing times varying between 7.5 and 240 seconds were studied. After mixing, specimens were prepared, each being compacted to the same density. After curing under sealed conditions for 7 days, specimens were tested in unconfined compression. All mixes increased in strength during the initial stages of mixing. They then reached a maximum strength from which they fell slightly as the mixing time was extended. The specimens with 9 percent liquid content reached their peak strength after only 15 seconds of mixing, while the mixture with only 6 percent emulsion required an optimum period of 30 seconds. It was apparent that mixing times in excess of optimum resulted in slight stripping, but the loss of strength was not great.

To investigate the mixing order, i. e., the order in which the ingredients should be fed into the mixer, 8 different combinations were studied (the letters S, E, W, and F denote sand, emulsion, water, and filler respectively and are arranged to represent the order of addition): SFEW, SFWE, SEWF, SEFW, SWFE, SWEF, SF/EW, and SW/F/EW. These 8 batches each containing the 7.5 percent Lion emulsion, 2.5 percent water, and 5 percent cement filler were mixed in a ½-cu ft concrete mixer for a total period of 1.5 minutes. The individual ingredients were added to the sand one at a time at intervals of about 10 seconds. All specimens were then compacted to the same density and cured for 7 days in a sealed condition before being tested. The results of the unconfined compression tests obtained for each of the 8 orders of mixing are shown for comparison in Figure 15. Each strength plotted represents the average from 4 tests. This demonstrates the important difference that the order of addition can make to the strength of otherwise identical mixes.

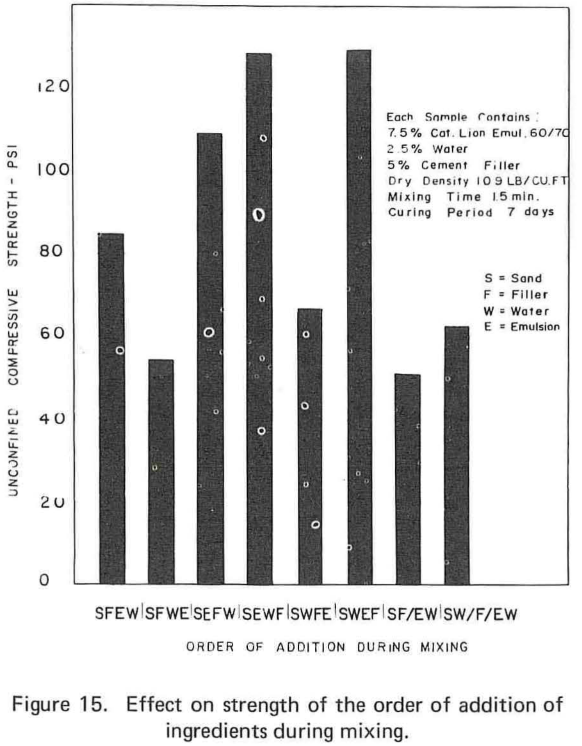

Figure 15. Effect on strength of the order of addition of ingredients during mixing.

It will be noted that the highest strengths were obtained by adding the filler last, while the lowest strengths were obtained by adding the cement filler first to dry sand. Presumably, the reason for this phenomenon is that, when the cement is added last, the bitumen has a chance to adhere to the surfaces of the sand particles before the cement is added. When the emulsion is added to a sand cement mix, the greater affinity of bitumen for the finer particles might result in poor coating of the sand.

CURING

During the initial stages of the investigation, specimens were compression-tested after being cured under sealed conditions for a period of 7 days at laboratory temperature. The strengths of these specimens were found to be very low, and it was appreciated that, when no active filler such as cement was to be added, a more appropriate method of curing would be to allow the specimens to dry out before testing. Similar conditions of air-drying would probably be more prevalent on construction sites than would soaked or sealed conditions.

A study of the effect of curing time was carried out by using specimens stabilized only with 6, 9, and 13 percent of RE11 emulsion with 60 to 70 penetration mixed with additional water contents of 4, 3.5, and 3 percent respectively. Compacted specimens were air-cured for periods varying between 1 and 12 weeks. Figure 16 shows their unconfined compressive strength plotted against bitumen content for different ages.

The interesting point to note is that, whereas at 7 days there appeared to be an optimum bitumen content of 5 percent, after 2 weeks the specimens with 7.4 percent bitumen had increased in strength more than those with lower bitumen contents, and the optimum bitumen content appeared to have increased. Hence, within the range of economic proportions of emulsion, the more emulsion that is added the higher the long term strength is likely to be under conditions of dry-curing.

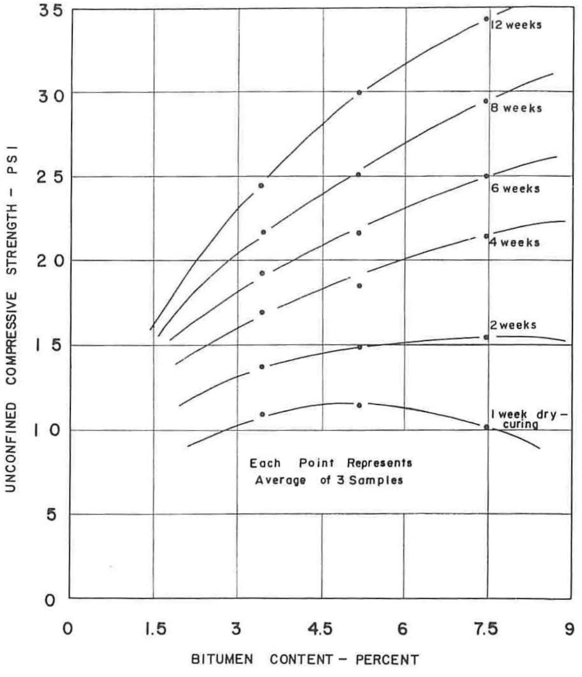

Figure 16. Effect of curing period on strength of emulsion stabilized soil.

CONCLUSIONS

When specimens of cationic emulsion-stabilized sand were triaxially tested after 7 days of dry-curing, an emulsion content in the range from 7 to 10 percent gave the highest cohesion, C_u. The angle of shearing resistance, ϕ_u, of the sand was reduced in proportion to the percentage of emulsion added but was independent of the viscosity of the base bitumen.

The presence of some water in the sand had the effect of increasing the ultimate 7-day strength of air-dried stabilized specimens. It was shown that as much water should be present during mixing as will not deleteriously affect compaction.

The unconfined compressive strength of sand-emulsion mixes was shown to be greatest when the emulsion was made with the lowest penetration grade of base bitumen.

The addition of filler to the uniformly graded sand had the effect of increasing the dry density achieved by a given amount of compactive effort and improved the mechanical stability of the sand. Cement filler greatly increased the compressive strength of sand-cement-emulsion mixes and was clearly acting more as a stabilizing agent than as a filler. These mixes, however, combined high strength without the accompanying brittleness of cement-stabilized material.

Other fillers such as hydrated lime, crushed limestone dust, and sand fines were effective in increasing strength, though to a lesser degree. This is supported by shear strength measurements in the field by Marais (10) who showed that sand-filler-emulsion mixes had strengths considerably higher than mixes lacking filler. It is recommended that, in circumstances where uniformly graded sand is to be stabilized with emulsion, a filler should be added if economically possible.

When a filler was added to a mix, it was demonstrated that the order in which ingredients were added affected the ultimate strength of the material. The highest strength is obtained by always adding the filler last after the other ingredients have been given a preliminary mix.

The compressive strength of emulsion-stabilized sand was shown to significantly increase with age. Tests on specimens cured for 3 months in air-dry conditions showed that (within the limit of economic proportions of emulsion) the more emulsion used, the higher the strength was.

ACKNOWLEDGMENTS

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Appendix

STANDARD PROCEDURE USED FOR SAMPLE PREPARATION

Mixing was carried out either in a ½-cu ft, 500-watt, pan type of concrete mixer (for large batches) or in a 150-watt Sunbeam food mixer. A known weight of oven-dry sand was fed to the mixer, and then water (if any), emulsion, and filler were added. Mixing was continued until no better coating could be achieved. Unless otherwise specified, 2-in. wide by 4-in. high split molds similar to those described in B.S. 1924 were used. The only difference was that an inner liner of cardboard or perspex tube was

used, so that a 2-in. diameter specimen enclosed in a protective cover could be extruded from the mold. This prevented disintegration of the specimens during extrusion.

Compaction was by means of a vibrating Kango type L hammer with a 2-in. nominal diameter plunger. The hammer weighs 15 lb, has 400-watt power, and has a frequency of 2,100 cycles/minute. The standard procedure was to apply 20 seconds of vibration to each of the 3 equal layers. For some experiments, however, as explained earlier, the amount of compaction was adjusted by trial and error to produce a specimen of a required density.

During compaction, the remaining mix was covered to prevent loss by evaporation. On completion of compaction, the nuts clamping the molds were slackened, and the liner tubes containing the samples were extracted. When specimens were wet-cured, the lining tubes were sealed top and bottom and stored for 7 days at 18 C. Before testing, the liners were split down the side and the samples released intact.

Where specimens were dry-cured, they were ejected from perspex liners out of which they were more easily released than from standard split molds. The specimens were then left to cure by exposure to the laboratory atmosphere for 7 days, after which they were weighed, measured, and tested.

ESTIMATIONS OF INDIRECT TENSILE STRENGTHS FOR LIME-TREATED MATERIALS

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Within the framework of a research effort concerned with the evaluation of tensile properties of stabilized subbase materials, it was desired to establish a means to predict indirect tensile strength from Texas Highway Department tests for lime-treated materials. Experiments were designed to develop a predictive equation in terms of factors involved with mix design, construction, and curing and to determine if acceptable correlations exist between the indirect tensile test and both the cohesiometer test and the unconfined compression test. These correlations were developed by using 2 approaches: The first varied 5 factors (compactive effort, molding water content, lime content, curing temperature, and clay content) at 3 levels each, and the second, based on Texas Highway Department testing procedures, varied only molding water content, lime content, and clay content because curing temperature and compactive effort are fixed by the test specifications. It was found that acceptable correlations for lime-treated materials exist for both approaches between the indirect tensile test and cohesiometer test, the indirect tensile test and unconfined compression test, and the indirect tensile test and the combined results of the cohesiometer and unconfined compressive test. These correlations provide the capability of estimating the indirect tensile strength from the cohesiometer or the unconfined compression test data or both for lime-treated materials used in pavements now in service.

•THE IMPORTANCE of the tensile characteristics in a rational design procedure for subbases can be demonstrated from both theoretical considerations and field observations. Nevertheless, until recently, little attention has been given to the tensile characteristics of stabilized materials and, therefore, little information was available. For this reason, the Center for Highway Research at the University of Texas at Austin began a research project to study the tensile properties of stabilized subbase materials for use in pavement design. On the basis of a review of existing methodology and laboratory tests, Kennedy and Hudson (1, 2) concluded that the indirect tensile test was the best test currently available for the evaluation of the tensile characteristics of stabilized materials.

In order to obtain information on the tensile strengths of lime-treated materials, 2 sequential experiments were conducted to determine the factors and the interactions among the factors that were important to tensile strength. A secondary objective was to develop a preliminary regression equation that could be used to estimate tensile strength in terms of the factors investigated. In addition, an attempt was made to determine whether correlations exist between the indirect tensile test and the tests used by the Texas Highway Department for evaluating lime-treated materials. Currently, the unconfined compression test and cohesiometer test are used by the Texas Highway Department.

The primary purpose of these correlation studies was to establish a means by which indirect tensile strengths could be estimated from unconfined compressive strengths or

Sponsored by Committee on Lime and Lime-Fly Ash Stabilization and presented at the 50th Annual Meeting.

cohesiometer values because these tests were used to evaluate lime-treated materials used in pavements in Texas for which performance records have been maintained. Any subbase design procedure must consider performance, and these records can be used in conjunction with the correlations to estimate tensile strengths of the materials at the time of construction and, thus, to obtain performance information without waiting for test sections to be approved, funded, and constructed.

EXPERIMENT DESIGN

A central composite rotatable design was used to evaluate the effects produced by the 5 factors investigated and to develop predictive equations by a regression analyis. This design consisted of a 2^5 full factorial with 32 cells, 10 star points, and 6 center points (48 observations). The full factorial in this design allowed analysis of the main effects and of all interaction effects on the tensile strength of lime-treated materials for the factors and levels studied. The star points and center points allowed analysis of the curvilinear effects. The replicate center points also provided an estimate of experimental error. This basic design was utilized in the development of a regression equation capable of estimating indirect tensile strengths from the factors and conditions included in this experiment and for the conditions of the study. The factors and levels selected are given in Table 1.

The general correlation experiment consisted of a half fraction of a 2^5 factorial with 16 observations plus 3 center points for each of the 3 tests. The 5 factors were the same as those used in developing the regression equation. The factors and levels are given in Table 2.

Only 3 of the factors could be varied in the Texas Highway Department correlation because the Texas Highway Department standard procedures fixed the compactive effort and the curing temperature; thus, a design involving a 2^3 full factorial with 8 cells, 6 star points, and 6 center points was used. The factors and levels are given in Table 3.

In both correlation experiments, 3 companion specimens were prepared for each treatment combination: a 2-in. high by 6-in. diameter specimen to be tested in indirect tension, a 2-in. high by 6-in. diameter specimen to be tested in the cohesiometer, and an 8-in. high by 6-in. diameter specimen to be tested in unconfined compression.

TABLE 1

FACTORS AND LEVELS FOR THE REGRESSION ANALYSIS

Factor	Level				
	-2	-1	0	+1	+2
A, compactive effort (4), psi	75	100	125	150	175
D, molding water content, percent	8.0	10.5	13.0	15.5	18.0
E, lime content, percent	0.0	1.5	3.0	4.5	6.0
F, curing temperature, deg F	50	75	100	125	150
H, clay content, percent	25.0	37.5	50.0	62.5	75.0

TABLE 2

FACTORS AND LEVELS IN THE GENERAL CORRELATION

Factor	Level		
	-1	0	+1
A, compactive effort (4), blows per layer	50.0	75.0	100.0
D, molding water content, percent	10.5	13.0	15.5
E, lime content, percent	1.5	3.0	4.5
F, curing temperature, deg F	75.0	100.0	125.0
H, clay content, percent	37.5	50.0	62.5

TABLE 3

FACTORS AND LEVELS IN THE TEXAS HIGHWAY DEPARTMENT CORRELATION

Factor	Level				
	-1.682	-1	0	+1	+1.682
D, molding water content, percent	8.8	10.5	13.0	15.5	17.2
E, lime content, percent	0.477	1.5	3.0	4.5	5.523
H, clay content, percent	29.0	37.5	50.0	62.5	71.0

PROPERTIES OF MATERIALS

The lime used in the experiments was a hydrated calcitic lime manufactured by the Austin White Lime Company. Its chemical composition, determined by the Texas Highway Department laboratories is as follows:

Chemical	Percent by Weight
$Ca(OH)_2$	93.67
CaO	0.0
Free water content, H_2O	1.38
$CaCO_3$	3.75
Inert matter such as SiO_2	1.20
Residue retained on No. 30 (590-micron) sieve	0.0

The aggregate used in the experiments was a rounded, pit-run gravel known locally as Seguin gravel. It was quarried near Seguin, Texas, and is used in south central Texas as a base material. Its properties are as follows:

Unified classification	GM_d
Wet ball mill	37.2
Los Angeles abrasion	
100 revolutions	7.3
50 revolutions	27.3

The clay used in the experiments is common to central Texas and is known as Taylor marl clay. Its mineralogical and plasticity characteristics are as follows:

Characteristic	Percent
Calcium montmorillonite	30 to 35
Illite	50 to 60
Kaolinite	10 to 15
Liquid limit	59
Plastic limit	18
Plastic index	41

PREPARATION AND CURING OF SPECIMENS

For the tests, the aggregate was separated and recombined to meet the gradation requirement shown in Figure 1. However, Taylor marl clay was substituted for all material finer than the No. 40 sieve. The specimen preparation and curing for the 3 phases of the experiment are summarized in the following paragraphs.

Regression Analysis

All regression analysis specimens were compacted by Texas gyratory shear compaction. Although compactive efforts could not be calculated, relative compactive

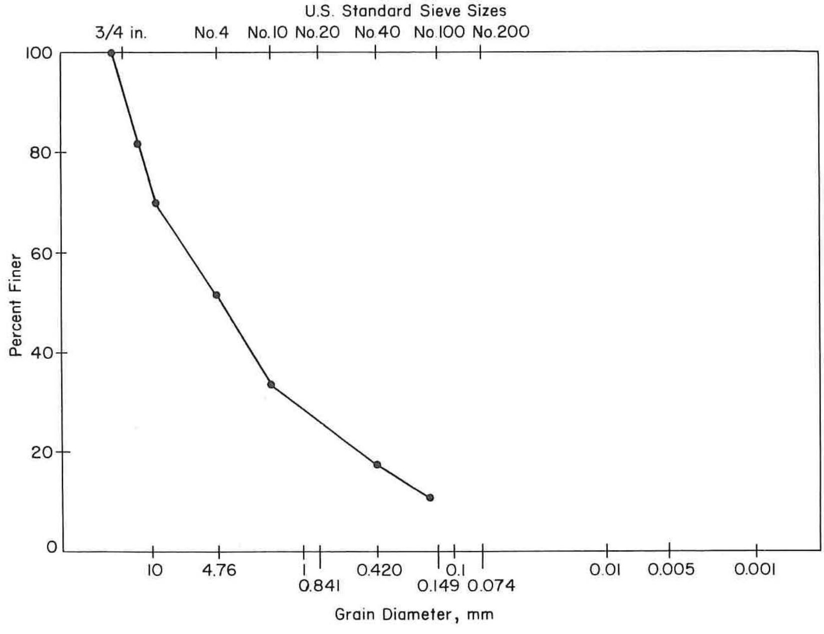

Figure 1. Grain size distribution for medium gradation.

efforts were specified in terms of the resistance the specimens produced on the hydraulic ram. High resistance pressures represented higher compactive efforts. Following compaction, the specimens were weighed and measured and were then wrapped in PVC film and cured 21 days at the specified curing temperatures given in Table 1.

General Correlation Analysis

The general correlation was an attempt to relate cohesiometer and unconfined test results with indirect tensile strengths obtained from specimens selected from the full factorial design. All specimens were impact compacted with a Rainhart compactor. The compactive efforts, given in Table 2, were specified as a number of blows per layer struck by a 10-lb hammer falling 18 in. After compaction, the specimens were weighed, and the height and the diameter or circumference were measured. The specimens were then wrapped with a layer of PVC film, placed in the appropriate temperature environment, and cured for 3 weeks prior to testing.

Texas Highway Department Correlation Analysis

The Texas Highway Department specifies that impact compaction be done by using a 10-lb ram with an 18-in. drop and subjecting each layer of lime-treated material to 50 blows. The curing procedure specified for lime-treated materials (5) is as follows: (a) The test specimens are extruded from the mold with the top and bottom porous stones in place, immediately covered with a triaxial cell, and then stored at room temperature for 7 days; (b) after the moist-curing period, they are removed from the cells and dried at a temperature not exceeding 140 F for about 6 hours or until one-third to one-half of the molding moisture is removed; and (c) they are cooled for at least 8 hours, weighed, measured, and then placed in triaxial cells and subjected to capillarity for 10 days with a constant lateral pressure of 1 psi and a surcharge weight of 15 lb.

TEST PROCEDURE

The procedure followed for the indirect tensile testing of soil-lime specimens was the same as that originally recommended by Kennedy and Hudson (1, 2) and the same as that reported by Tullock et al. (4). Essentially, the test consists of the application of compressive loads along a diametrical plane (Fig. 2). These loads result in a tensile stress distribution that is perpendicular to and along the plane containing the applied lime loads and that causes failure by splitting along this plane.

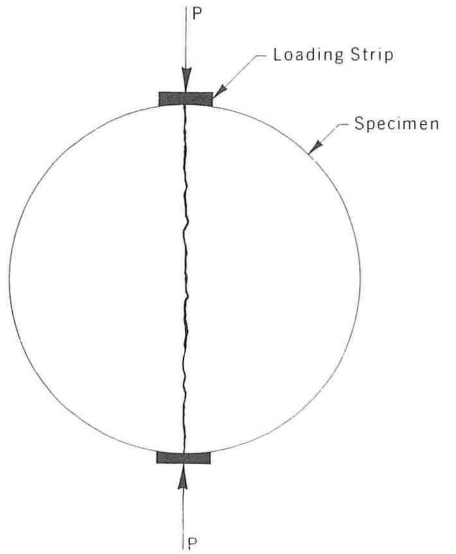

Figure 2. Indirect tensile test.

Testing was conducted at 75 F at a loading rate of 2 in./min. Each specimen had a nominal diameter of 4 or 6 in. and a nominal height of 2 in. A loading strip with a curved portion having a radius of 3 in. was used to test the 6-in. diameter specimens, and one with a curved portion having a radius of 2 in. was used to test the 4-in. diameter specimens. More detailed descriptions of the test are given in other reports (1, 2, 4).

The unconfined compression tests and the cohesiometer tests were conducted according to Texas Highway Department standard procedures (5). The cohesiometer specimens were tested at the laboratories of the Texas Highway Department. The unconfined compression test specimens had a nominal diameter of 6 in. and a nominal height of 8 in., and the cohesiometer specimens had a nominal diameter of 6 in. and a nominal height of 2 in. The test temperature for all specimens was 75 F. The loading rate for the unconfined compression test was 0.05 in./min, and the cohesiometer loading rate was 1,800 ± 20 grams/min of shot.

ANALYSIS

Regression Equation

A regression analysis was conducted in order to obtain an equation with which to estimate the indirect tensile strengths of lime-treated materials for the conditions of the experiment. The levels of the factors used in the experiment are given in Table 1. The resulting predictive equation was

$$\begin{aligned}\hat{S}_t = {} & 228.18 - 1.647A + 3.100D - 86.375E - 2.218F - 5.234H + 0.017AF \\ & + 0.035AH + 0.581AE + 0.043FH + 0.137DH + 1.727EH - 0.037DF \\ & + 0.929EF - 0.261D^2 - 0.611E^2 + 0.0028F^2 - 0.008H^2 - 0.0116AEH \\ & - 0.0058AEF - 0.000348AFH - 0.0173EFH + 0.000116AEFH \qquad (1)\end{aligned}$$

where

$\hat{S}_t$ = predicted value of indirect tensile strength, psi; and
A, D, E, F, H = factors considered for prediction (Table 1).

The multiple correlation coefficient for the predictive equation was 0.94, and the standard error of estimate was ±4.03 psi.

The regression equation utilizes the uncoded factor levels given in Table 1. The coded levels, i.e., -2, -1, 0, +1, and +2, should not be used in the calculation of estimated indirect tensile strengths. Because this regression is based on the uncoded

levels, the equation is valid only for predictive purposes and cannot be interpreted term by term. It should also be noted that the predictive capabilities of the regression are valid only for the conditions of this study and the factors and levels included in the study. The use of any levels outside of this factor space is not recommended and caution is required for intermediate levels within the experiments.

Correlation Analysis

The ultimate objective of the correlation analysis was the development of relationships with which to estimate indirect tensile strengths of lime-treated materials when the unconfined compressive strengths or cohesiometer values are known.

General Correlation—Plots of indirect tensile strengths versus unconfined compressive strengths and cohesiometer values are shown in Figures 3 and 4 respectively. A regression analysis was run on the data, and the following equations were obtained:

$$\hat{S}_t = 16.46 + 36.7q_u \tag{2}$$

for which the multiple correlation coefficient was 0.89 and the standard error of estimate was ±5.9 psi;

$$\hat{S}_t = 7.46 + 2.19\ (C/100) \tag{3}$$

for which the multiple correlation coefficient was 0.93 and the standard error of estimate was ±4.8 psi; and

$$\hat{S}_t = 9.27 + 14.8q_u + 1.46\ (C/100) \tag{4}$$

for which the multiple correlation coefficient was 0.94 and the standard error of estimate was ±4.4 psi,

where

$\hat{S}_t$ = predicted value of indirect tensile strength, psi;
q_u = measured value of unconfined compressive strength, ksi; and
C = measured cohesiometer value, in grams/in. of width corrected to a 3-in. height.

Texas Highway Department Correlation—Plots of indirect tensile strengths versus unconfined compressive strengths and cohesiometer values are shown in Figures 5 and 6 respectively. A regression analysis was conducted, and the following prediction equations were obtained:

$$\hat{S}_t = -1.43 + 96.5q_u \tag{5}$$

for which the multiple correlation coefficient was 0.85 and the standard error of estimate was ±2.4 psi;

$$\hat{S}_t = 1.52 + 4.59\ (C/100) \tag{6}$$

for which the multiple correlation coefficient was 0.75 and the standard error of estimate was ±3.0 psi; and

$$\hat{S}_t = -1.68 + 74.4q_u + 1.6\ (C/100) \tag{7}$$

for which the multiple correlation coefficient was 0.87 and the standard error of estimate was ±2.3 psi,

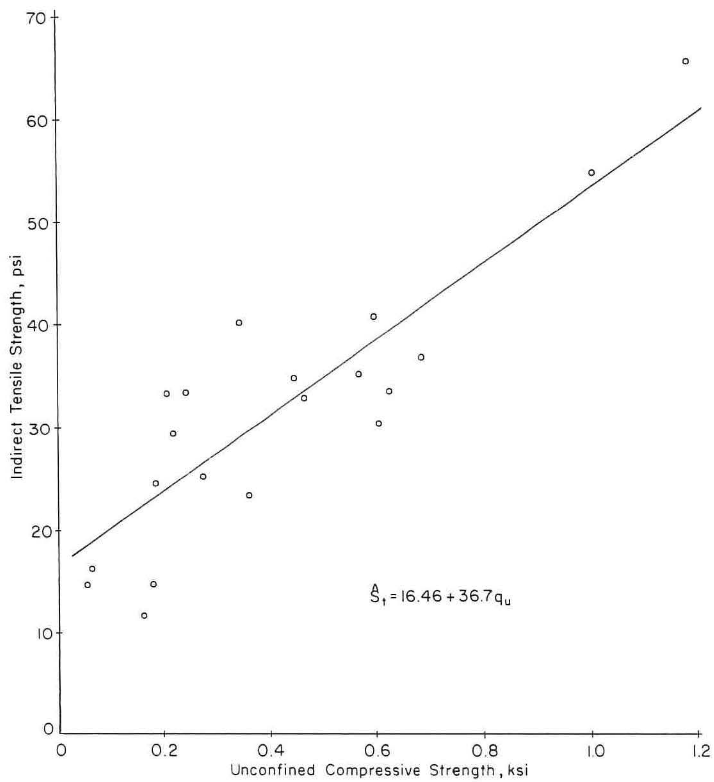

Figure 3. Relationship of indirect tensile strength and unconfined compressive strength for general correlation.

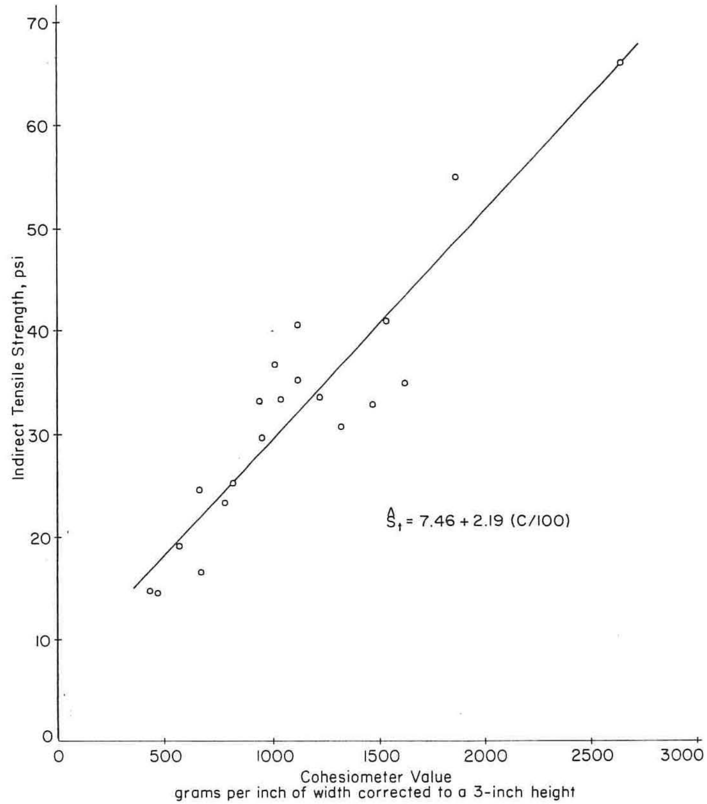

Figure 4. Relationship of indirect tensile strength and cohesiometer value for general correlation.

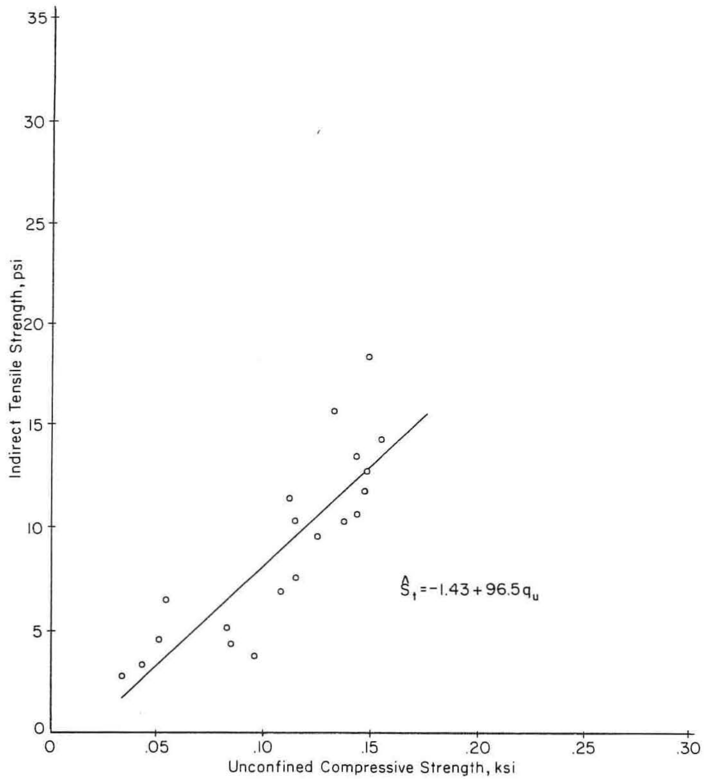

Figure 5. Relationship of indirect tensile strength and unconfined compressive strength for Texas Highway Department correlation.

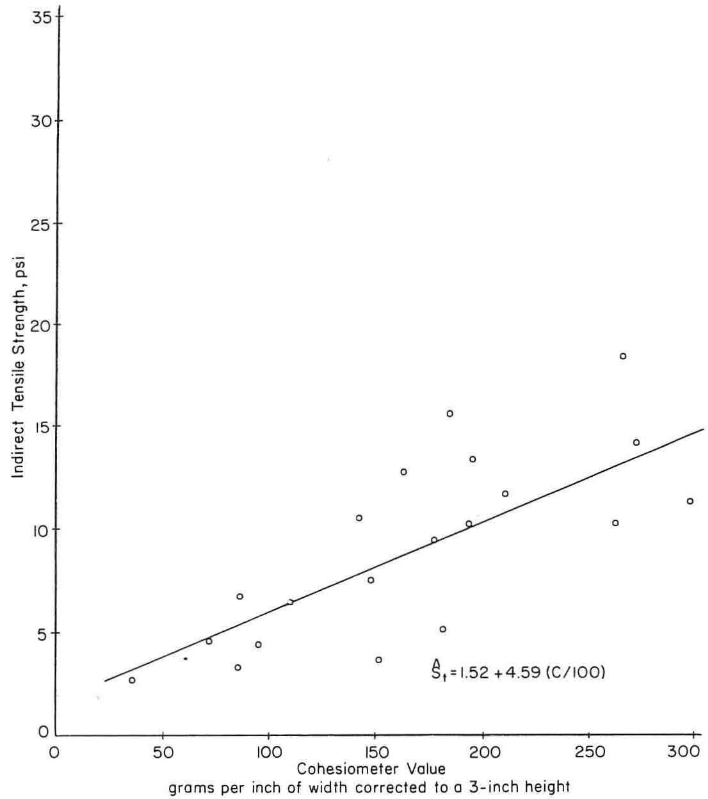

Figure 6. Relationship of indirect tensile strength and cohesiometer value for Texas Highway Department correlation.

where

$\hat{S}_t$ = predicted value of indirect tensile strength, psi;
q_u = measured value of unconfined compressive strength, ksi; and
C = measured cohesiometer value, in grams/in. of width corrected to a 3-in. height.

Combined Correlation—The strengths of the specimens tested for the Texas Highway Department correlation were generally less than the strengths of those tested for the general correlation. The Texas Highway Department correlation specimens were cured in capillarity for 10 days prior to testing, which probably accounts for their lower strengths. Because the ranges of strength of the 2 correlations were quite different, the data from the experiments were combined to check for a relationship between indirect test results of the unconfined compression test and the cohesiometer test over the entire range of strengths. Figures 7 and 8 show the combined data. A regression analysis was run on these combined data, and the following prediction equations were obtained:

$$\hat{S}_t = 6.89 + 50.6q_u \tag{8}$$

for which the multiple correlation coefficient was 0.91 and the standard error of estimate was ±6.4 psi;

$$\hat{S}_t = 5.52 + 2.33\ (C/100) \tag{9}$$

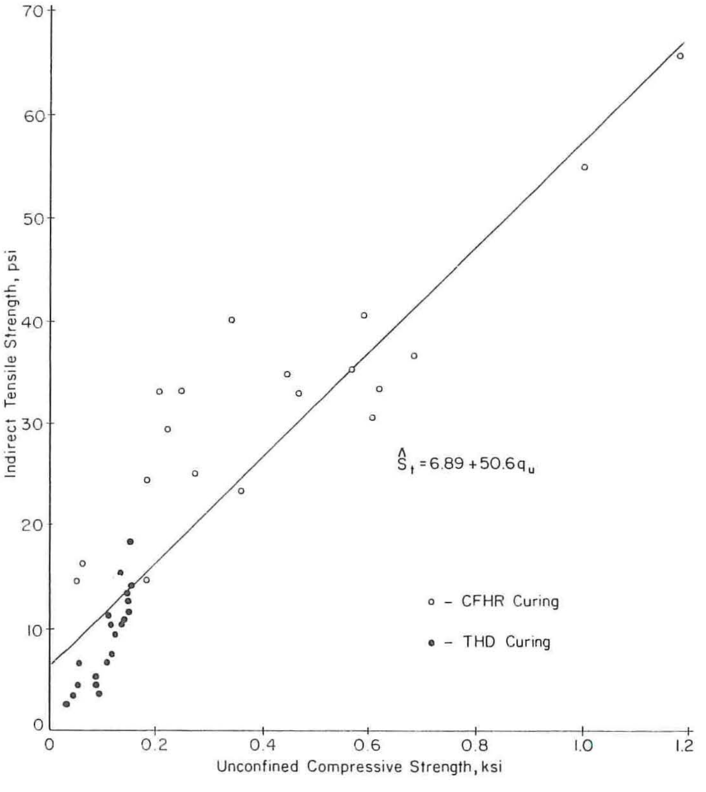

Figure 7. Relationship of indirect tensile strength and unconfined compressive strength for combined correlation data.

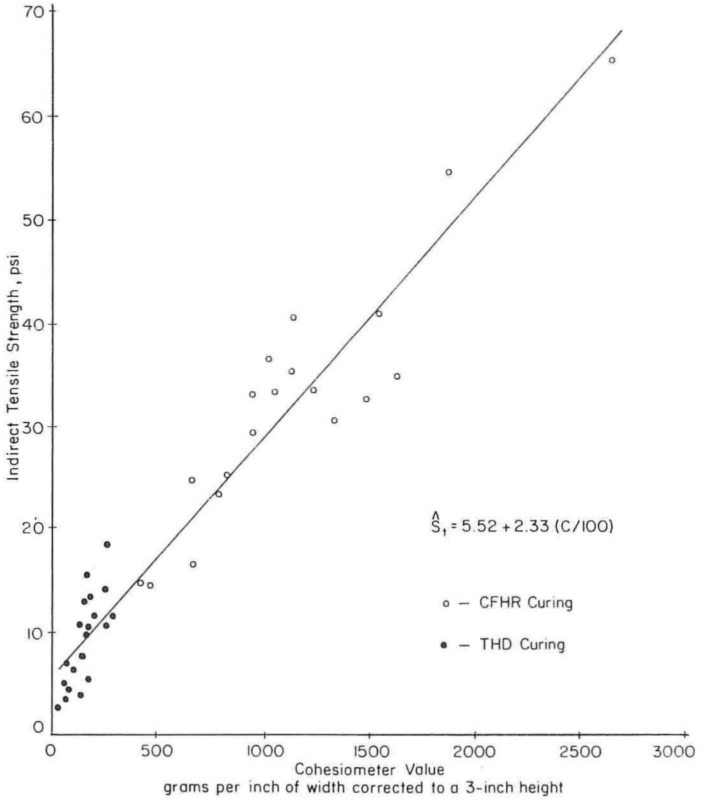

Figure 8. Relationship of indirect tensile strength and cohesiometer value for combined correlation data.

for which the multiple correlation coefficient was 0.96 and the standard error of estimate was ±4.1 psi; and

$$\hat{S}_t = 3.61 + 16.5q_u + 2.3\ (C/100) - 0.03\ (C/100)^2 \qquad (10)$$

for which the multiple correlation coefficient was 0.97 and the standard error of estimate was ±3.7 psi,

where

$\hat{S}_t$ = predicted value of indirect tensile strength, psi;
q_u = measured value of unconfined compressive strength, ksi; and
C = measured cohesiometer value, in grams/in. of width corrected to a 3-in. height.

DISCUSSION OF RESULTS

A summary of the correlation studies is given in Table 4. Whenever an experiment is designed for studying the possible correlation between 2 materials tests, criteria are required for judging the acceptability of the results from both a statistical and engineering viewpoint.

The first test used to determine whether there were correlations was based on the multiple correlation coefficients. A minimum value for R^2 of 0.25 was selected. At

TABLE 4

CORRELATION SUMMARY

Correlation Variables	Multiple Correlation Coefficient	Does Correlation Exist?	Standard Error of Estimate (psi)	Coefficient of Variation	Is Correlation Acceptable?
Tensile strength versus unconfined compressive strength					
General	0.89	Yes	±5.9	0.18	Yes
Texas Highway Department	0.85	Yes	±2.4	0.26	Yes
Combined	0.91	Yes	±6.4	0.31	Yes
Tensile strength versus cohesiometer					
General	0.93	Yes	±4.8	0.15	Yes
Texas Highway Department	0.75	Yes	±3.0	0.33	Yes
Combined	0.96	Yes	±4.1	0.20	Yes
Tensile strength versus cohesiometer and unconfined compressive strength					
General	0.94	Yes	±4.4	0.14	Yes
Texas Highway Department	0.87	Yes	±2.3	0.25	Yes
Combined	0.97	Yes	±3.7	0.18	Yes

this level, as outlined in Snedecor and Cochran (6), the correlation coefficient R for all the correlations presented is significant at a probability level of 95 percent. In fact, they are all significant at the 99 percent probability level, and it is believed with confidence, therefore, that all the correlations presented do exist.

The second test used to determine the acceptability of the correlation data considered the standard error of estimate. A large standard error is unacceptable because the tensile strengths of lime-treated materials are relatively low, but the decision as to whether the standard error is too large must be left to the judgment of the user. The largest standard error obtained for the various correlations was ±6.4 psi, which was considered to be within tolerable limits because the indirect tensile test for the lime-treated materials used in the experiments had a standard error of ±3.05 psi for duplicated specimens (4).

It is interesting to note (Figs. 7 and 8) that the restrictions of the Texas Highway Department procedures provided results that were confined to a specific range, an outcome which might result from any test procedure that severely restricts the manner in which the material can be varied. Most materials tests attempt to reproduce the most severe conditions under which the materials will have to perform satisfactorily in the field in order to provide conservative results. However, doing so may require procedures that result in an inference space so narrow that results are completely unrealistic. This indicates that materials testing concepts, practices, and theories must be changed if maximum information is to be obtained for design purposes.

SUMMARY AND CONCLUSIONS

This paper presents a predictive equation that, for the factors and levels of factors investigated, predicts indirect tensile strength reasonably well for the conditions of this experiment. In addition, correlations relating indirect tensile strength with both cohesiometer and unconfined compressive test results were developed and evaluated. It was shown that correlations exist for these tests and that tensile strengths may be estimated from cohesiometer and unconfined compressive strength data within some given level of error.

ACKNOWLEDGMENTS

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TENSILE PROPERTIES FOR DESIGN OF LIME-TREATED MATERIALS

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Center for Highway Research, University of Texas at Austin

The design procedures for lime-treated materials have often been empirical in nature and are oriented for subgrade stabilization of modification. At the present time, design methods are based primarily on unconfined compressive strengths, plasticity characteristics of the soil or binder, or pH values. Design procedures for lime-treated subbase materials are very limited in scope, and the performance records of these materials used in in-service rigid pavements in Texas have generally been poor. This paper presents the results of a study that investigated factors affecting the indirect tensile strength of lime-treated materials. The factors studied were clay content, lime content, molding water content, compactive effort, and curing temperature. Significant effects produced by these factors and their interactions are discussed. A regression equation and a design table are presented for estimating indirect tensile strengths in terms of the factors investigated. The design table enables the engineer to estimate the tensile strengths for proposed lime-treated mixtures that are designed by using currently accepted design procedures.

•DURING the past 5 years, research has been conducted at the University of Texas at Austin to evaluate the tensile properties of subbases for use in rigid pavement design. A review of the literature indicated a lack of well-documented information concerning tensile properties of stabilized materials. Therefore, an experimental program was conducted to obtain such information. This paper summarizes the findings from laboratory studies on lime-treated subbase materials and discusses their application to mixture design.

A preliminary study (1) was designed to determine the important factors affecting the tensile strength of lime-treated materials and to determine the nature of the effects. Table 1 gives the 8 factors and their respective levels included in this initial study, which was made as a guide for future work. A second experiment (2) involved a more detailed analysis and evaluation of the effects of 5 of the original 8 factors on the tensile strength of lime-treated materials.

PROPERTIES OF MATERIALS

Lime used in the experiments was a hydrated calcitic lime manufactured by the Austin White Lime Company. The following chemical composition was determined by Texas Highway Department laboratories:

Chemical	Percent by Weight
$Ca(OH)_2$	93.67
CaO	0.0
Free water content, H_2O	1.38
$CaCO_3$	3.75
Inert mattter such as SiO_2	1.20
Residue retained on No. 30 (590-micron) sieve	0.0

Sponsored by Committee on Lime and Lime-Fly Ash Stabilization and presented at the 50th Annual Meeting.

TABLE 1

FACTORS AND LEVELS SELECTED FOR PRELIMINARY STUDY

Factor	Level					Variable Type
	Low		Medium		High	
A, compactive effort[a]	Low				High	Quantitative
B, compaction type[a]	Impact				Gyratory shear	Qualitative
C, curing procedure	Air-dried				Sealed	Qualitative
D, molding water content, percent by weight	8	10	12	16	20	Quantitative
E, lime content, percent by weight	2		4		6	Quantitative
F, curing temperature, deg F	40		75		110	Quantitative
G, curing time, weeks	2		4		6	Quantitative
H, clay content, percent by weight	15		50		100	Quantitative

[a]Compactive effort for both types of compaction is given in Table 2.

The aggregate used in the experiments was a rounded, pit-run gravel known locally as Seguin gravel. It was quarried near Seguin, Texas, and is used in south central Texas as a base material. Its properties are

Unified classification	GM_d
Wet ball mill	37.2
Los Angeles abrasion	
100 revolutions	7.2
50 revolutions	27.3

The clay used in the experiments consists primarily of illite and montmorillonite. It is common to the central Texas area and is locally known as Taylor marl clay. Its plasticity and mineralogical characteristics are as follows:

Characteristic	Percent
Liquid limit	59
Plastic limit	18
Plastic index	41
Calcium montmorillonite	30 to 35
Illite	50 to 60
Kaolinite	10 to 15

EXPERIMENT DESIGN AND PROCEDURES

The 5-factor experiment was selected as the basis for the recommendations and conclusions in this paper because it was a more detailed study, founded on the results from the preliminary study. The factors and levels studied are given in Table 3.

A central composite rotatable experimental design (3) provided an economical and practical means of studying the effects produced by the 5 factors and their interactions with a minimum number of observations. This design consisted of a 2^5 full factorial with 32 cells, 10 star points, and 6 center points. The center points were replicate specimens that were produced by combining the middle or "zero" levels of all factors. The full factorial provided data for the analysis of the effects produced by the 5 factors and their

TABLE 2

TYPES OF COMPACTION

Compaction Type	Compactive Effort	
	Low	High
Impact, ft-lb/in.^{3}	21	45
Gyratory shear[a], psi	75	125

[a]Compaction procedure for the Texas gyratory shear compactor (9).

TABLE 3

FACTORS AND LEVELS FOR ADVANCED EXPERIMENT

Factor	Level					Variable Type
	-2	-1	0	1	2	
A, compactive effort[a], psi	75	100	125	150	175	Quantitative
B, compaction type			Gyratory shear			Qualitative
C, curing procedure			Sealed			Qualitative
D, molding water content, percent by weight	8	10.5	13	15.5	18	Quantitative
E, lime content, percent by weight	0.0	1.5	3.0	4.5	6.0	Quantitative
F, curing temperature, deg F	50	75	100	125	150	Quantitative
G, curing time, weeks			3			Quantitative
H, clay content, percent by weight	25.0	37.5	50.0	62.5	75.0	Quantitative

[a]Compaction procedure for the Texas gyratory shear compactor (9).

interactions; the star points and center points enabled the curvilinear effects to be evaluated.

The indirect tensile test (Fig. 1) consists of applying opposite compressive loads along the vertical diametral plane of a cylindrical specimen, resulting in a relatively uniform tensile stress perpendicular to and along the diametral plane containing the applied load. Failure usually occurs as splitting along this loaded plane when the tensile stress exceeds the tensile strength of the material.

The procedure followed for the indirect tension testing of lime-treated subbase specimens was that originally recommended by Hudson and Kennedy (4, 5) modified slightly (2).

TABLE 4

INDIRECT TENSILE STRENGTHS

Specimen	Level of Factor[a]					Indirect Tensile Strength (psi)
	A	E	H	D	F	
Full Factorial						
1	+1	+1	+1	+1	+1	42.7
2	+1	+1	+1	+1	-1	30.8
3	+1	+1	+1	-1	+1	35.2
4	+1	+1	+1	-1	-1	22.6
5	+1	+1	-1	+1	+1	33.3
6	+1	+1	-1	+1	-1	17.8
7	+1	+1	-1	-1	+1	53.8
8	+1	+1	-1	-1	-1	27.1
9	+1	-1	+1	+1	+1	22.8
10	+1	-1	+1	+1	-1	23.3
11	+1	-1	+1	-1	+1	24.6
12	+1	-1	+1	-1	-1	19.7
13	+1	-1	-1	+1	+1	17.5
14	+1	-1	-1	+1	-1	15.8
15	+1	-1	-1	-1	+1	40.5
16	+1	-1	-1	-1	-1	18.8
17	-1	+1	+1	+1	+1	26.9
18	-1	+1	+1	+1	-1	31.4
19	-1	+1	+1	-1	+1	27.4
20	-1	+1	+1	-1	-1	17.4
21	-1	+1	-1	+1	+1	37.1
22	-1	+1	-1	+1	-1	15.4
23	-1	+1	-1	-1	+1	53.7
24	-1	+1	-1	-1	-1	23.6
25	-1	-1	+1	+1	+1	20.1
26	-1	-1	+1	+1	-1	19.1
27	-1	-1	+1	-1	+1	22.0
28	-1	-1	+1	-1	-1	15.8
29	-1	-1	-1	+1	+1	12.5
30	-1	-1	-1	+1	-1	11.6
31	-1	-1	-1	-1	+1	27.7
32	-1	-1	-1	-1	-1	17.5
Star Points						
33	-2	0	0	0	0	24.2
34	+1	0	0	0	0	38.2
35	0	-2	0	0	0	22.6
36	0	+2	0	0	0	23.4
37	0	0	-2	0	0	18.7
38	0	0	+2	0	0	28.1
39	0	0	0	-2	0	23.2
40	0	0	0	+2	0	20.8
41	0	0	0	0	-2	25.6
42	0	0	0	0	+2	45.5
Center Points						
43	0	0	0	0	0	22.0
44	0	0	0	0	0	29.3
45	0	0	0	0	0	28.9
46	0	0	0	0	0	27.9
47	0	0	0	0	0	26.0
48	0	0	0	0	0	30.4

[a]Factors and levels are given in Table 3.

Testing was conducted at 75 F at a loading rate of 2 in./min. The specimens had nominal diameters of 4 or 6 in. and nominal heights of 2 in. A loading strip with a curved portion having a radius of either 3 in. or 2 in. was used to test the 6-in. diameter and 4-in. diameter specimens. The experimental results are given in Table 4.

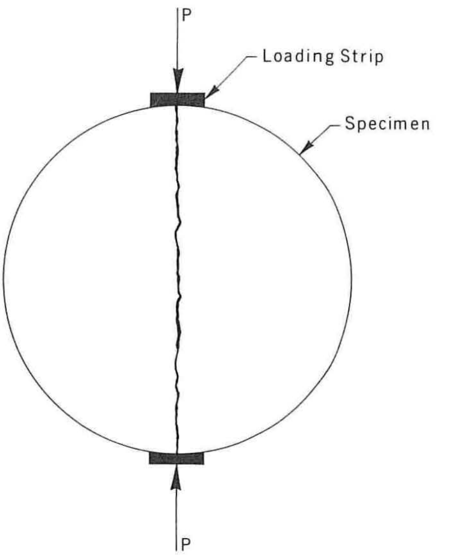

Figure 1. The indirect tensile test.

DISCUSSION OF RESULTS

In the following sections, effects produced by the various factors and their interactions and the curvilinear effects that were shown to be highly significant from both a statistical and engineering viewpoint are discussed. Although it is not possible from this experiment to explain definitely the observed effects, it is possible to postulate their probable causes.

Interactions

Many times the effect produced by varying the levels of a factor is dependent on the level of other factors. These interaction effects are very important and often reflect the complex interrelationships among factors affecting the behavior of a material. Four 2-factor interactions, which were statistically significant at the 5 percent confidence level, were judged to have engineering importance and are discussed in the following.

Curing Temperature and Clay Content—For a curing temperature of 75 F, a change in clay content from 37.5 to 62.5 percent caused a slight increase in indirect tensile strength. However, for a curing temperature of 125 F, the same change in clay content caused a considerable decrease in the indirect tensile strength. This loss was probably due to cracking of the high clay content specimens cured at elevated temperatures.

Molding Water Content and Clay Content—Figures 2, 3, and 4 show the trends predicted from the experimental strength data and the importance of this estimation. Estimated indirect tensile strengths were plotted against molding water content for lime percentages of 2, 4, and 6 percent. For the lower clay percentages, the majority of the specimens were on the wet side of an optimum strength. As the clay percentage increased, the observations centered around the optimum water content strength for the middle clay contents and fell on the dry side of the strength-water content curves for the high clay percentages. Therefore, as the lime content was increased, strength increased for the higher levels of clay content but decreased for small percentages of clay. Thus, an increase in lime content did not necessarily result in an increased strength after a given curing period.

Another interpretation of this interaction is that, when a low water content was combined with a low clay content, there was sufficient water to allow the strength-gaining reaction to take place. With a low water content and an increased clay content, however, there was an insufficient water content for the soil-lime reactions to take place because of absorption by the clay particles. Such a possibility is supported by the fact that, when the water content in combination with the high clay content was increased from 10.5 to 15.5 percent, there was an accompanying strength increase. However, when the water content in combination with the low clay content was increased from 10.5 to 15.5 percent, there was a sharp decrease in tensile strength because the molding water content was on the wet side of optimum.

Molding Water Content and Curing Temperature—At the molding water content of 10.5 percent, an increase in curing temperature from 75 to 125 F caused a marked increase in indirect tensile strength. However, for the specimens compacted at a water content

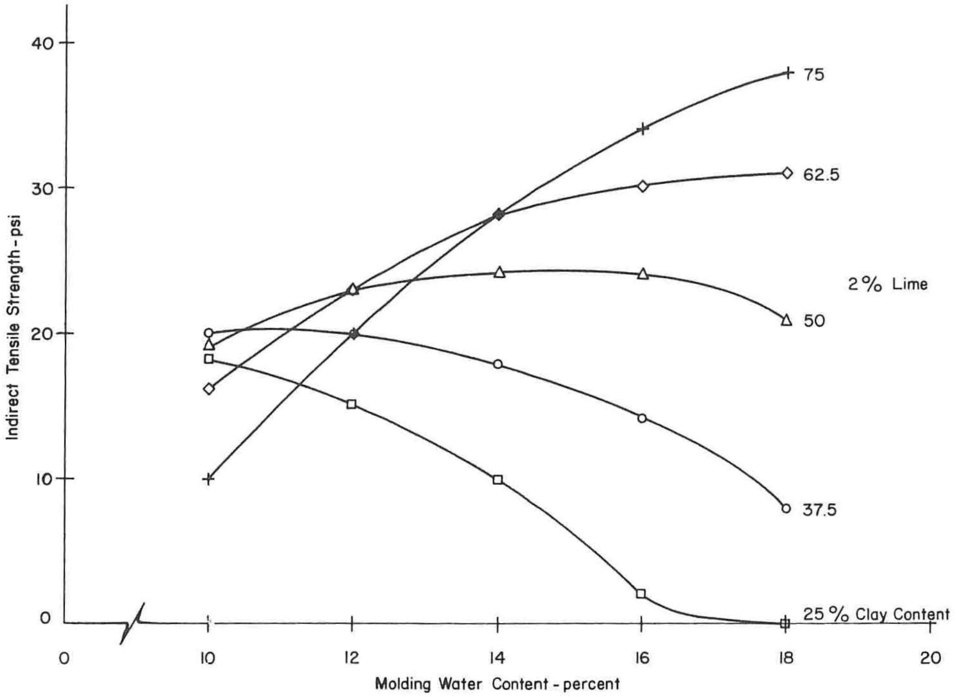

Figure 2. Effect of molding water content and clay content on indirect tensile strength of soil-aggregate mixtures containing 2 percent lime.

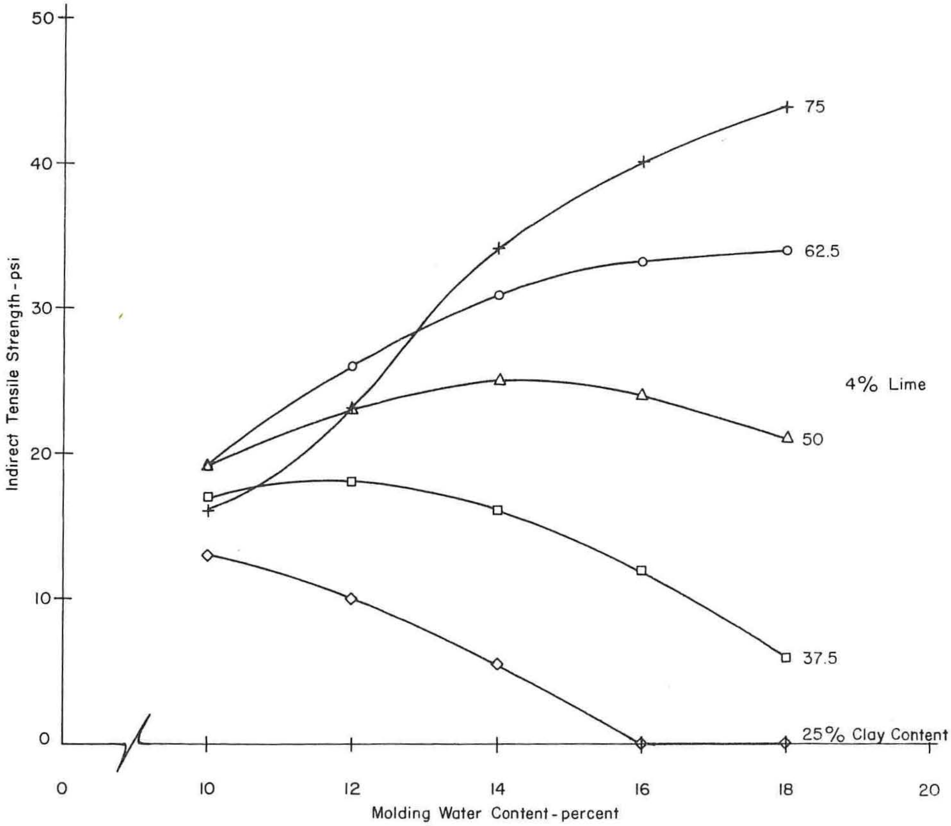

Figure 3. Effect of molding water content and clay content on indirect tensile strength of soil-aggregate mixtures containing 4 percent lime.

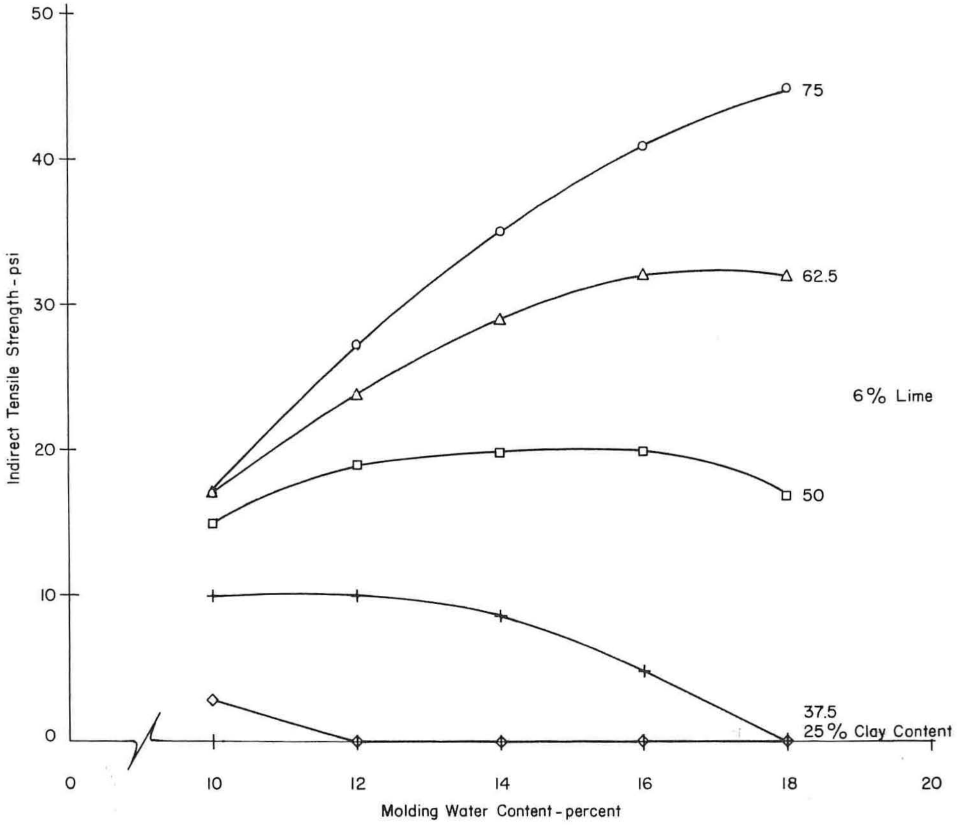

Figure 4. Effect of molding water content and clay content on indirect tensile strength of soil-aggregate mixtures containing 6 percent lime.

of 15.5 percent, the strength increase was much less for the same increase in curing temperature. Because 15.5 percent was on the wet side of optimum and excessive water caused a reduction in the strength of the clay matrix of the specimens, the strength increase due to an increase in curing temperature was less apparent than it was in the low water content specimens, which were relatively dry and hard.

Lime Content and Curing Temperature—For specimens with a lime content of 1.5 percent, an increase in curing temperature from 75 to 125 F caused an increase in the indirect tensile strength of the specimens; but for specimens with a lime content of 4.5 percent, the same increase in curing temperature caused a much greater increase in specimen strength. It is probable that at the low lime content there was insufficient lime for the increased curing temperature to have much effect.

Main Effects

The analysis of variance showed that 4 of the factors produced significant effects at the 5 percent probability level and that 3 factors were significant at the 1 percent level. In addition, it was found that 2 of these main effects were curvilinear. Clay content was the only factor that did not appear to be important. It was found that the average indirect tensile strength was increased by increasing the compactive effort, increasing the lime content, decreasing the molding water content, and increasing the curing temperature. The latter 2 factors, curing temperature and molding water content, produced significant curvilinear or quadratic effects.

Curing Temperature—Tensile strength increased with an increased curing temperature; the increase associated with raising the temperature from 100 to 150 F was much greater than that associated with raising the temperature from 50 to 100 F. This observation is supported by Ruff and Ho (5), who reported a greater rate of strength increase associated with a temperature increase in the higher temperature ranges.

Molding Water Content—The average indirect tensile strength increased when the molding water content was increased from 8 to 13 percent but decreased when it was raised from 13 to 18 percent. Thus, there was an average optimum water content for the materials tested and conditions of the tests.

Prediction Equation

A regression analysis was conducted in order to obtain an equation with which the indirect tensile strength of the lime-treated materials in this study could be estimated. The use of this prediction equation is valid only for the range of levels of the factors considered and the conditions in this experiment. The prediction equation derived from the experiment is

$$\begin{aligned} S_t = {} & 228.18 - 1.647A + 3.100D - 86.375E - 2.218F - 5.234H + 0.017AF \\ & + 0.035AH + 581AE + 0.043FH + 0.137DH + 1.727EH - 0.037DF \\ & + 0.929EF - 0.261D^2 - 0.611E^2 + 0.0028F^2 - 0.008H^2 - 0.0116AEH \\ & - 0.0058AEF - 0.00348AFH - 0.0173EFH + 0.000116AEFH \end{aligned} \tag{1}$$

where

S_t = predicted value of indirect tensile strength, in psi, and
A, D, E, F, H = factors considered (Table 2).

The coefficient of determination R^2 for the predictive equation was 0.88, and the standard error of estimate was ±4.03 psi.

The regression equation utilizes the uncoded factor levels given in Table 3 and not the coded levels, i.e., -2, -1, 0, +1, and +2. Because this regression is given for the uncoded data, the equation is valid only for predictive purposes and cannot be interpreted term by term.

APPLICATION TO DESIGN

At present there are few definite design procedures for lime-treated materials and no method of designing lime-treated mixtures for a specified or desired strength. Eades (6), McDowell (7), and Thompson (8) have formulated guidelines for the determination of the percentage of lime required for lime-treated materials.

Eades based his procedure on the pH of the lime-soil mixture, which is a measure of the amount of lime consumed by the soil. Nevertheless, it was noted that strength gains vary with the mineralogical components of a soil and that a strength test is necessary to determine the percentage of increase.

McDowell, on the other hand, developed a chart based on performance and field measurements that expresses the percentage of lime required if the percentage of binder in the soil-aggregate mixture and the plasticity index of the binder are known. McDowell stated that use of the chart did not eliminate the need for materials tests and recommended that strength tests be conducted to verify the acceptability of the lime percentages obtained from the chart. It was also recommended that a soil-aggregate mixture treated with lime should have a minimum unconfined compressive strength of 50 psi when used as a subbase or subgrade.

Thompson suggested that unconfined compression tests samples be made by using the natural soil and the soil treated with lime. It was recommended that the lime-treated soil be allowed to cure for 48 hours at 120 F. Then, if the strength increase of the lime-treated soil is less than 50 psi compared with the untreated soil, the soil is considered to be nonreactive; if the strength difference between the natural soil and the lime-treated soil is greater than 50 psi, the soil is termed reactive. The nonre-

active soil is then treated to obtain a minimum plasticity index, and the reactive soil is reevaluated to obtain a lime content that produces a maximum unconfined compressive strength.

With the exception of these references to strength, there is little additional information available regarding the design of lime-treated materials in terms of strength or load-deformation characteristics. Thus, it would be desirable to develop information that would allow a lime-treated mixture to be designed to provide a given level of strength or that would allow an estimate of its strength to be made. Such information would supplement these procedures and would provide a basis for the development of a rational mixture design procedure. Such a procedure is desirable for the design of pavements to achieve an optimum pavement section in terms of strength and economic requirements.

It is, therefore, proposed that the predictive equation previously discussed be used as a means of estimating tensile strengths of lime-treated mixtures similar to the mixtures used in this study and possibly as a basis for the development of a design procedure. Although the equation can be solved directly, a table of tensile strengths has been generated in terms of clay content, moisture content, curing temperature, and lime content (Table 5). The purpose of the data given in Table 5 is to expedite the estimation of tensile strengths without the need for extensive manual calculations or the use of the computer.

The dry densities of the lime-treated specimens ranged between 110 and 125 pcf for all the treatment combinations tested. Within this density range, no correlation was

TABLE 5

INDIRECT TENSILE STRENGTH, PSI, BASED ON TEMPERATURE AND CLAY, LIME, AND MOISTURE CONTENTS

Lime (percent)	Moisture (percent)	25 Percent Clay			37.5 Percent Clay			50 Percent Clay			62.5 Percent Clay			75 Percent Clay		
		40 F	60 F	80 F	40 F	60 F	80 F	40 F	60 F	80 F	40 F	60 F	80 F	40 F	60 F	80 F
0	10	21	18	18	20	17	16	17	14	13	11	8	7	2	0	0
	12	20	15	13	22	18	15	22	17	15	19	14	12	14	9	7
	14	16	10	6	22	16	12	25	19	15	26	19	16	24	17	13
	16	10	3	0	19	12	6	26	18	13	30	22	17	31	24	18
	18	2	0	0	15	6	0	25	16	9	32	23	16	37	28	21
1	10	19	19	21	20	19	20	19	17	17	15	12	12	8	5	4
	12	17	16	16	22	19	19	24	21	20	23	19	17	20	15	13
	14	14	10	10	22	18	16	27	22	20	30	24	21	30	24	19
	16	8	3	1	19	14	10	28	22	18	34	27	22	37	30	24
	18	0	0	0	15	8	3	27	19	13	36	28	21	43	34	26
2	10	16	18	23	19	20	23	20	19	21	18	16	16	13	10	9
	12	14	15	18	20	20	22	25	23	23	26	23	21	25	20	17
	14	10	10	12	20	18	18	28	24	23	32	28	25	35	28	24
	16	4	2	3	18	14	13	28	24	21	37	30	26	42	34	29
	18	0	0	0	13	8	6	27	21	17	39	31	25	48	38	31
3	10	11	16	24	16	19	24	19	20	23	19	18	19	17	13	12
	12	9	13	19	18	20	23	24	23	25	28	25	24	29	24	21
	14	6	8	12	18	18	20	27	25	25	34	30	28	38	32	28
	16	0	1	4	15	14	14	28	24	13	38	32	29	46	38	32
	18	0	0	0	11	8	7	27	22	19	40	33	28	52	42	35
4	10	5	13	23	12	17	25	17	19	24	20	19	20	19	16	14
	12	4	10	18	24	18	23	22	23	26	28	26	26	31	26	23
	14	0	5	12	14	16	20	25	25	25	34	31	29	41	34	30
	16	0	0	3	12	12	15	26	24	24	39	33	30	48	40	34
	18	0	0	0	7	6	7	25	21	20	41	34	30	54	44	37
5	10	0	8	21	8	14	24	14	18	24	19	19	21	20	17	15
	12	0	6	17	9	15	22	20	21	26	27	26	26	32	27	24
	14	0	0	10	9	13	19	22	23	26	34	30	30	42	35	31
	16	0	0	1	6	9	14	23	22	24	38	33	31	50	42	35
	18	0	0	0	2	3	6	22	20	19	40	34	30	55	46	38
6	10	0	3	8	1	10	21	10	15	22	17	17	20	20	17	15
	12	0	0	14	3	10	20	15	19	24	25	24	25	32	27	24
	14	0	0	7	3	9	17	18	20	24	32	29	29	42	35	31
	16	0	0	0	0	5	12	19	20	22	36	32	30	50	41	35
	18	0	0	0	0	0	4	18	17	18	38	32	29	55	45	38

found to exist between tensile strength and dry density. However, it is stressed that a lime-treated mixture must be well compacted for the lime reaction to occur and for the development of strength.

The design table has treatment combinations that have estimated "zero" tensile strength. Because these mixtures have low clay contents and the aggregate is nonreactive to lime stabilization, this lack of tensile strength would be expected. Although lime does not produce strength improvements with these mixtures, it does favorably modify the plasticity characteristics of the clay binder to greatly decrease its swelling potential. Often this benefit can be substaintial.

CONCLUSION

The information on the interaction effects among the factors studied indicates that the mechanisms of lime stabilization are complex in nature and cannot be modeled adequately in simple terms. The documentation of these effects gives greater understanding of the tensile behavior of lime-treated materials.

In addition, information is provided in the form of a predictive equation that can be used to estimate tensile strength or to begin to provide preliminary mixture designs for materials similar to those studied. As additional tests are conducted on similar and different materials stabilized with either lime or another additive, the equation can be modified and expanded. Thus, by incorporating strength requirements, cost of the additive and the placement of the treated materials, and estimates of tensile strength either from a direct test or possibly a predictive equation such as the regression equation given here, the designer will be able to make a better decision as to the type of additive and the amounts needed to sufficiently upgrade a substandard construction material.

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EFFECT OF SOIL SURFACE AREA AND EXTRACTABLE SILICA, ALUMINA, AND IRON ON LIME STABILIZATION CHARACTERISTICS OF ILLINOIS SOILS

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Several years ago, a group of about 40 Illinois soils was collected by M. R. Thompson of the University of Illinois. Thompson determined the lime reactivity of the soils and also subjected the samples to a number of routine analytical procedures in an attempt to determine if lime reactivity is closely related to any particular engineering, mineralogical, or physicochemical property of the soil. The writers sought to extend Thompson's analytical work to include determination of several other soil properties not included in the original work. Specific properties investigated included soil surface area, amount of finely divided iron present in the soil, and amount of silica and alumina entering an aqueous solution maintained at the pH level of a saturated solution of lime in water. Work was done by using Thompson's original soil samples, and results obtained by the writers were compared with his determination of lime reactivity of the soils. It was found that lime reactivity does not appear to be very closely correlated with soil surface area and even less so with the availability of dissolved silica or alumina or both. However, it was found that lime reactivity does appear to be significantly inhibited by the presence of finely divided iron intimately distributed through the soil mass, a condition that is typical of many better drained soils in Illinois.

• CONSIDERABLE INTEREST has arisen in recent years in the improvement of the engineering properties of natural soil materials through lime treatment. Several years ago, a representative group of Illinois soils was collected by M. R. Thompson of the University of Illinois. Thompson subjected these soils to lime treatment in the laboratory and determined the effect of treatment on unconfined compressive strength. The soils were all essentially surface materials and included samples of A-, B-, and C-horizons of some 14 pedological series. Some deeper occurring parent materials (loesses and glacial tills) were also included in the study.

Soils were treated with varying percentages of lime (3, 5, and 7 percent), compacted at optimum water content by a procedure similar to the standard AASHO specification, and cured under moist conditions for 28 and 56 days. Specimens were then tested in unconfined compression, and the results were compared with strengths of untreated but similarly processed specimens of the same materials.

An attempt was made by Thompson to relate the effect of lime treatment on soil strength with a number of soil properties that were routinely determined for all the soils that he used. Determined engineering properties included grain-size distribution and Atterberg limits. Mineralogical properties included determination of predominant clay minerals present and determination of the $CaCO_3$ equivalent for calcareous

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materials. Physicochemical properties included pH, percentage of organic carbon present, cation exchange capacity, total exchangeable bases, and specific amounts of Ca, Mg, Na, and K. It was found that organic carbon content exerted a significant adverse effect on lime reactivity. A significant positive correlation appeared to exist between lime reactivity and soil pH, percentage of base saturation of the soil, and presence of mixed-layer minerals in the clay fraction.

The authors became interested during the summer of 1969 in possible physicochemical explanations for lime reactivity. Thompson provided samples of 24 of the soils used in his study (Table 1).

It has been concluded by several investigators, including Thompson and Eades (3, 8), that the strength increase accompanying lime treatment of many soils results from a pozzolanic reaction. This would imply that the lime releases free silica or alumina or both from the minerals present in the soil and then reacts with the released material to form crystalline calcium silicates and aluminates that cement individual soil particles together. With this in mind, the authors concluded that lime reactivity of a soil should be strongly correlated with the amount of silica and alumina found to be present in a slurry made from a particular soil in which the pH is maintained at 12.3 for a given time. This is the pH presented by a saturated lime-water solution. It was also thought that high soil surface area should enhance lime reactivity because it would appear that

TABLE 1

LIME REACTIVITY AND CHEMICAL PROPERTIES OF 24 ILLINOIS SOILS

Soil Type	Horizon	Drainage Classification	Lime Reactivity (psi)	Percent Increase in STR With Lime	Surface Area (m^2/gm)	Extractable Fe_2O_3 (percent)	Extractable Al_2O_3 (percent)	Extractable SiO_2 (percent)
Bryce silty clay	B	Poor	131	162	178	1.47	0.21	0.57
Cisne silt loam	B	Poor	97	104	159	1.35	0.54	0.42
Cowden silt loam	B	Poor	43	74	191	1.79	0.30	0.37
Cowden silt loam	C	Poor	90	134	117	1.56	0.17	0.73
Cowden silt loam	CMC	Poor	90	134	104	1.64	0.17	0.32
Drummer silty clay loam	B	Poor	118	173	171	1.04	0.19	0.23
Elliott silt loam	B	Moderately well to imperfect	12	12	159	2.60	0.28	0.33
Fayette silt loam	B	Well	44	63	144	2.14	0.27	0.41
Fayette silt loam	C	Well	145	363	74	1.24	0.21	0.43
Hosmer silt loam	B_2	Well to moderately well	60	95	136	1.83	0.36	0.36
Hosmer silt loam	B'_2	Well to moderately well	37	42	174	1.91	0.27	0.45
Huey silt loam	B	Imperfect	131	128	134	1.43	0.17	0.33
Huey silt loam	C	Imperfect	133	149	124	1.74	0.14	0.45
Miami silt loam	B	Well	21	26	76	1.64	0.30	0.34
Miami silt loam	C	Well	98	134	82	1.58	0.08	0.23
Sable silty clay loam	B	Poor	127	130	187	0.92	0.17	0.21
Tama silt loam	B	Well to moderately well	24	32	191	1.37	0.26	0.35
Wisconsinan clay till			89	114	170	1.31	0.18	0.50
Wisconsinan loam till			79	75	48	1.22	0.11	0.41
Illinoian till			135	265	47	1.43	0.07	0.39
Peorian loess, calcareous			76	345	54	1.02	0.09	0.46
Peorian loess, leached			24	72	72	1.52	0.17	0.31
Buried Illinoian profile	B	Poor	93	216	75	0.73	0.08	0.40
Accretion gley profile	G	Very Poor	230	328	100	0.24	0.35	0.86

Note: Drainage classification and lime reactivity taken from Thompson (8, Tables 3 and 8). Surface area and extractable Fe_2O_3, Al_2O_3, and SiO_2 were determined by authors using Thompson's samples.

more silica and alumina would be released in a given treatment under such conditions. It was also thought that a correlation between the presence of iron oxide and lime reactivity might exist because several investigators (4, 10) have proposed that in well-drained soils iron oxide takes the form of tiny positively charged molecular aggregations that adhere to the surface of clay minerals rendering them less vulnerable to attack by materials present in the surrounding soil solution.

METHODS AND PROCEDURES

Routine tests performed on the 24 samples included the following:

1. Soils were treated in a sodium hydroxide solution in which the pH was adjusted to 12.3. Two grams of soil were placed in 20 ml of solution. The resulting slurries were placed in 4-oz polyethylene bottles and shaken continuously for 3 days. Following this, the slurries were centrifuged, and the resulting clear solutions were subjected to analysis for silica and alumina. The method proposed by Ingamells was used in silica analysis (6). Alumina content was determined by the aluminon lake procedure discussed in several publications (5, 6). The writers experienced some difficulty with this procedure, and some comment regarding the analysis should probably be made at this point. Best results were obtained by adding 3 ml of 1N HCl to a 1-ml aliquot of solution. This acidulated solution was then heated for 1 hour at 80 C to ensure that all aluminum present would be in the form of Al^{+++} ions without associated hydroxyl ions. The solution was then built to 50 ml by the addition of a buffer solution presenting a pH of 4.2 prepared from acetic acid and sodium acetate. Five milliliters of aluminon solution, also buffered at a pH of 4.2, were then added. The solution was then shaken, and the aluminum content was determined colorimetrically by using a Coleman Junior spectrophotometer 20 min after the addition of the aluminon solution. This time period did not vary by more than 1 min and yielded good results for a number of standard solutions of various concentrations. It was found that the solution tends to darken and become turbid if allowed to stand for a longer period of time before the colorimetric determination.

2. Surface area of the soils was determined. Ethylene glycol monoethyl ether was used instead of the more traditional ethylene glycol. It has been demonstrated that the ether provides good results with a considerably shorter time requirement for equilibration (2).

3. Iron was extracted from the samples by using sodium citrate and sodium dithionate in a process described by Aguilera and Jackson (1). Iron dissolved from the soil was determined colorimetrically by the thiocyanate method. This iron-removal procedure results in the solution of the finely divided iron oxy-hydroxides that may be present as coatings on clay particles. However, concretionary iron and iron ions that may have proxied for other ions in the mineral structure of the clays are not attacked and dissolved.

RESULTS

Somewhat to the surprise of the authors, it was found that only a fair correlation exists between surface areas of the soils studied and the response of the soils to lime stabilization treatment. Perhaps even more surprising was the fact that this correlation is an inverse one: Soils presenting a high surface area exhibited poorer response to treatment. A possible explanation may be drawn from Eades' work (3) in which it was found that montmorillonitic soils appear to require the addition of a certain percentage of lime before strengthening effects are noted and that response occurs with only slight additions of lime to soils containing other clay minerals. However, in Thompson's study (8) many of the soils presenting high surface areas, notably those of the Cowden, Bryce, Tama, and Sable series, responded best to a 5 percent lime treatment. Strengths obtained by using either a 3 or 7 percent treatment were lower.

At this point it may be well to note that, in most Illinois soils, the clay fraction is predominantly made up of 3-layer minerals: illite in the case of glacial tills and montmorillonite in the case of loessial soils. Strictly local conditions may alter the pattern

just described. A locally acid environment appears to result in the release of aluminum from the 3-layer, clay-mineral structure and the subsequent formation of chlorite or mixed-layer clays (4).

A considerable variation in content of extractable silica and alumina was found to exist in the group of soils analyzed. However, as Figure 1 shows, correlation of these properties with strength increase resulting from lime treatment does not appear to be highly significant. The inverse effect of alumina was particularly surprising. Eades (3) has suggested that bonds resulting from the formation of calcium aluminate cementing agents develop in a relatively short time but that such bonds are ultimately much weaker than those developed by the more slowly formed calcium silicate cements. A comparison of Thompson's 28- and 56-day strengths of lime-treated soils in which extractable alumina is notably more abundant than extractable silica, or vice versa, does not appear conclusive with regard to this concept.

It was found that a rather significant inverse correlation appears to exist between the extractable iron content of a soil and its response to lime stabilization treatment as represented by the percentage increase in unconfined compressive strength.

The primary mineral sources of iron in Illinois soils are oxides (principally magnetite and ilmenite) and silicates (including amphiboles and pyroxenes). Fragments of these minerals have been incorporated into the several glacial tills and loesses that cover most of the state. Also present is some iron that is substituted for aluminum in

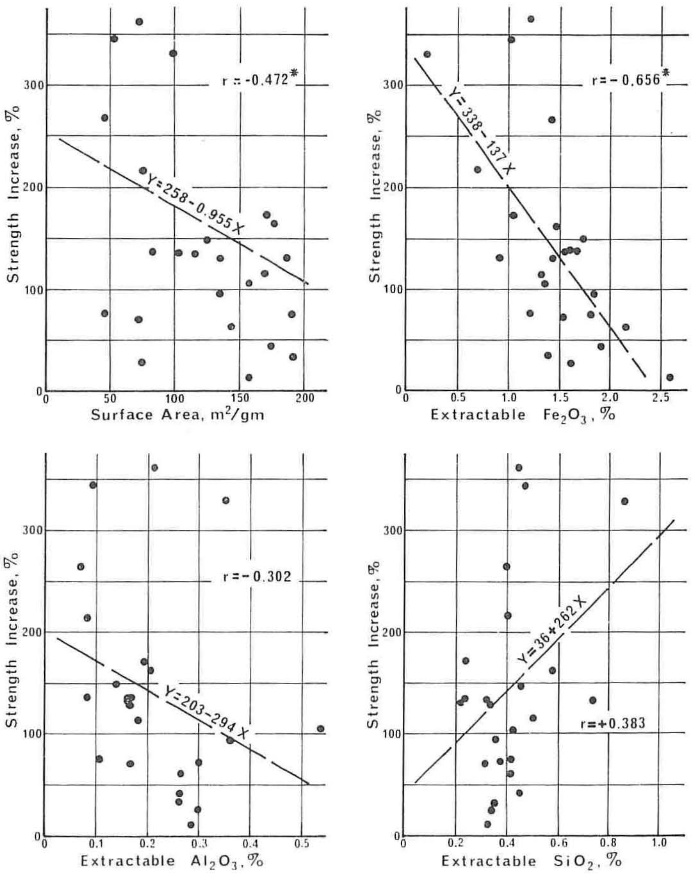

Figure 1. Relationship between increase in compressive strength with lime treatment and several physicochemical properties of 24 Illinois soils.

the crystalline structure of clay minerals. The primary iron-bearing minerals are subjected to chemical weathering processes in which soil microorganisms play an important role. Chemical change and redistribution of the iron within the soil mass occur. The nature of the redistribution is largely dependent on soil drainage conditions. Poorly organized oxy-hydroxides and goethite are the principal secondary minerals formed.

In poorly drained soils, the pH and oxidation potential of the environment have been such that much of the iron has been reduced to the Fe^{++} valence state. In this state the iron may be dissolved by the soil water and, hence, is quite mobile. Whereas some of the iron is no doubt leached from the soil, most of it moves a short distance within the soil mass to a nearby location where the microenvironment is favorable to oxidation and, hence, precipitation with formation of concretions. Eventually, most of the iron present in the soil becomes concentrated in these locations, and the bulk of the soil mass will contain very little iron.

In better drained soils, iron is moved through microscopically small distances by chemical and microbial weathering agents. However, the pH and oxidation potential of the environment do not favor the reduction of iron, and the iron present does not experience a period during which macroscopic relocation occurs. Thus, secondary iron minerals in various states of hydration occur in microaggregations, perhaps containing only a few molecules each. These aggregations usually present a net positive surface charge and, thus, are attracted to clay mineral particles that usually bear a negative surface charge. In the well-drained soils, weathering tends to result in an intimate scattering of iron throughout the bulk of the soil mass. It may be seen that the clay fraction of a well-drained soil may lose such properties as cation exchange capacity as clay particles become permanently plated by insoluble and chemically stable iron aggregations.

The contrast in iron distribution between poorly drained and well-drained soils is reflected in soil color. The subsoil of a typical well-drained profile is usually of a uniform yellow-brown color reflecting the intimate scattering of iron throughout the soil mass. Imperfectly drained soils may present a variegated gray and yellow mottled appearance whereas, in even more poorly drained soils, the soil mass is of a more or less uniform gray color and contains small hard concretions of dark reddish-brown limonitic material as well as limonitic linings along root or worm holes. The gray color is indicative of reduced presence of iron throughout the bulk of the soil mass.

It has been observed that the total iron content in most Illinois soils as may be found by fusing or dissolving a sample is relatively constant from place to place and usually totals about 4 to 5 percent Fe_2O_3. It would appear from the foregoing discussion, however, that the form in which the iron exists in the soil is of more concern than the total amount. The iron extraction method of Aguilera and Jackson (1) is well suited to this end in that iron contained in the small aggregations of a well-drained soil is dissolved whereas the concretionary iron of poorly drained soils is not attacked to any great extent. Thus the "extractable iron" found by this method is primarily iron that exists in the form of coatings on individual clay mineral particles. Determination of the magnetic susceptibility of a soil also provides a measure of the amount of iron present in the form of small aggregations and clay particle coatings because this iron is generally found in the form of a magnetic oxide. Limonitic or concretionary iron is predominantly goethite, which is far less magnetic (7).

From the fairly significant inverse correlation between extractable iron content of the soils analyzed and increase in strength of the soils with lime treatment, it would appear that successful lime stabilization is favored by the absence of finely divided iron oxy-hydroxides intimately associated with the clay fraction of the soil. Thus, it would be expected that poorly drained soils will exhibit greater lime reactivity than well-drained soils. This conclusion must, of course, be tempered by the realization that the poorly drained soils frequently contain higher percentages of organic carbon that has been shown to inhibit cementing action.

The rather poor correlations between lime reactivity and soil surface area and the availability of soluble silica and alumina in the soil would indicate that these properties are apparently not as important in the chemistry of lime stabilization treatment as the authors once thought they were.

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STRESS-DEFORMATION PREDICTION IN CEMENT-TREATED SOIL PAVEMENTS

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The applicability of elastic-layer theory and the finite-element method for the prediction of stresses and deflections in cement-stabilized soil pavements under repeated plate loads has been found to be good. Two pavement test sections, each 8-in. thick and 20 by 20 ft in plan were constructed by using a cement-treated silty clay overlying a clay subgrade. Pavement 1 contained 3 percent cement, and pavement 2 contained 6 percent cement. Instrumentation was developed and installed in the pavement that allowed measurement of vertical deflections both at the pavement surface and near the bottom of the stabilized layer, compressive stress at the top of the subgrade, and radial strain at the bottom of the stabilized layer. Characterization of material properties for use in analysis was done by using the results of strength and repeated load compression and flexure tests on undisturbed samples taken from the test sections. Because the results of this investigation demonstrate that stresses and strains in cement-stabilized soil pavements can be predicted successfully by using existing theory, a basis for pavement thickness design may be possible that limits critical stresses and strains within the pavement and subgrade to acceptable values.

•THE RESULTS of a survey recently completed by Fohs and Kinter (9) indicate that the current annual usage of cement-stabilized material in pavement structures in the United States averages about 50 million sq yd. This corresponds to approximately 3,500 miles of 24-ft wide roadway—a not inconsiderable amount of construction.

In spite of this large volume and the associated construction costs, most agencies base the thickness and quality design of cement-stabilized layers on empirical rules. Although it is likely that some degree of empiricism, or perhaps more properly reasoned adjustment of the design thickness, will always be required to account for factors not readily analyzable, improved methods of thickness design are needed.

The steps required for the development of an improved design technique have been listed (2) as follows:

1. Identification of loading and environmental conditions,
2. Characterization of material properties,
3. Establishment of failure criteria,
4. Stress and deformation analysis of a system representative of the pavement structure, and
5. Performance studies in the field for verification and modification of the proposed method.

A number of studies have provided information relative to steps 2 and 3, e.g., those by Felt and Abrams (8), Abrams (1), Bofinger (4), Shen and Mitchell (18), and Larsen and Nussbaum (13). Studies of field performance have been reported by Childs and

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Nussbaum (5), Highway Research Board (11), Larsen (12), Mitchell and Freitag (15), and Nussbaum and Larsen (17). Mitchell and Shen (16) have considered how a stress and deformation analysis (step 4) might be used as a basis for soil-cement thickness design. Larsen, Nussbaum, and Colley (14) have combined considerations of the load-deflection and fatigue behavior of soil-cement into a tentative design method for soil-cement pavements.

A field test program designed to provide information on the behavior of slabs of cement-stabilized soil under static and repeated loading has been completed. The test sections provided information useful for steps 2, 3, and 5 of the design process. Of particular interest in these tests was the direct measurement of stresses and strains at the bottom of the treated layer.

Studies have been made of the applicability of existing theory for the prediction of stresses and deflections in the pavement structure (step 4), and it is the purpose of this paper to describe these analyses and to compare prediction and observation.

TEST PAVEMENTS

Two test pavements each 20 by 20 ft in plan were constructed at the Richmond Field Station, University of California. The test pavements were essentially 2-layer systems each having an 8-in. thick cement-stabilized soil layer overlying the natural subgrade. A silty-clay soil was used in the stabilized layer, and the subgrade was a yellow clay. Classification properties of these soils are given in Table 1. The natural subgrade had a fairly uniform moisture content of about 20 percent and an average field CBR of about 8 to a depth at least 5 ft below the ground surface. Pavement 1 was constructed by using 3 percent cement by weight, and pavement 2 contained 6 percent cement. Construction details are presented by Wang, Mitchell, and Monismith (19).

Each test pavement had 9 different locations for repeated plate load tests. These 9 test sites were so distributed that the minimum spacing between the center of each site was 6 ft and the minimum distance from the center of any test site to the sides of the pavement was 4 ft as shown in Figure 1. These spacings, as verified from the test results, were sufficient to minimize the effects of pavement edges on the field performance as well as the repeated load tests on the soil properties at adjacent test sites.

LOADING FACILITIES AND INSTRUMENTATION

An 11,000-lb capacity loading piston actuated by compressed air was used to apply repeated loads at a frequency of 20 cycles/min and a duration of 0.1 sec. Reaction was provided by a beam that was fastened perpendicularly to the chassis at the end of a truck that carried 4 concrete blocks as reaction weight. During the test, the reaction beam was jacked up and supported by 2 wooden blocks. Photographs of the test setup are shown in Figures 2 and 3.

At each test site, 3 plates of different sizes were used, and a series of pressure intensities was applied to each plate in the sequence given in Table 2. In all cases the smallest plate was tested first, and the applied pressures were increased from the smallest value in order to minimize the prestress effects.

A stress gage of the diaphragm type was used for measuring vertical compressive stress at the top of the subgrade, a strain gage was used for measuring radial strain at the base of the cement-stabilizer layer, and dial gages were used to measure deflection at the top of the subgrade and at the surface of the pavement. The stress and strain gages were installed at the time of construction of the test pavements.

TABLE 1

SOIL CLASSIFICATION DATA

Property	RFS Silty Clay in Stabilized Layer	RFS Clay in Subgrade
Liquid limit, percent	29.2	56.3
Plastic limit, percent	19.4	22.6
Plastic index, percent	9.8	33.7
Specific gravity	2.65	2.66
Organic content, percent	2.5	1.3
Mineral composition of -2 μ fraction	Illite and montmorillonite	
Classification		
AASHO system	A-4	A-7
Unified system	CL	CH
Textural system	Clay loam	Clay

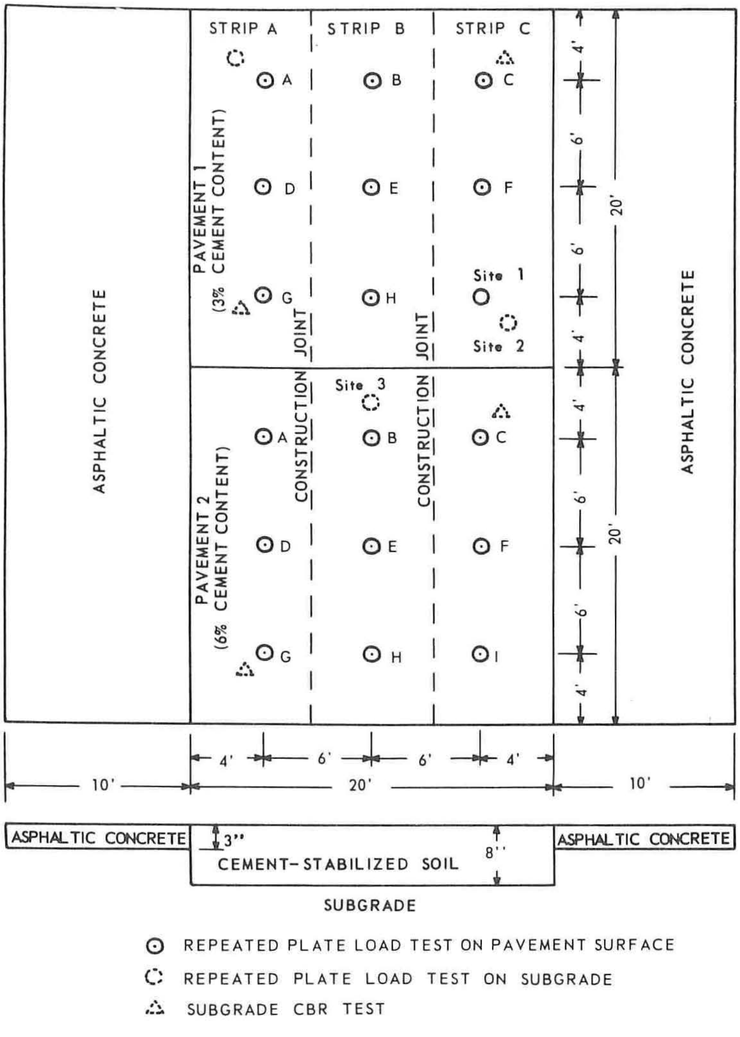

Figure 1. General layout of prototype pavements.

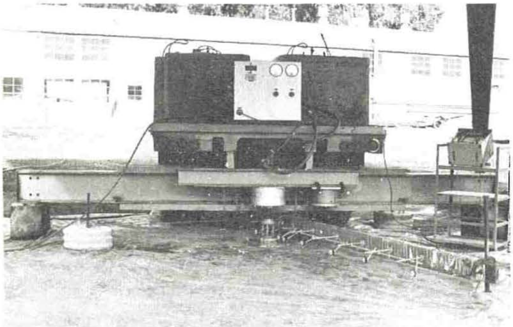

Figure 2. General view of repeated plate load test.

Figure 3. Close-up view of loading plate and gage system.

Gages were installed on the top of the subgrade at both sides of the center of loading plate in such positions that the center of the stress gage was 3 in. away from the center of the loading plate, whereas the central point of the strain gage was 2 in. off the centerline of the loading plate (Fig. 4). This arrangement was adopted to prevent the stress gage response from being affected by stress concentrations due to the presence of the strain gage and a $^7/_{16}$-in. diameter vertical hole used for measuring vertical deflection at the top of the subgrade. The effect of the release of vertical pressure at the bottom of the $^7/_{16}$-in. hole on the vertical stress acting on the stress gage was studied by using the approach suggested by Geddes (10), and it was found that the effect of pressure relief at a point 2 in. away and 1.5 in. from the bottom of the $^7/_{16}$-in. hole was insignificant.

TABLE 2

PLATE SIZES AND PRESSURE INTENSITIES USED IN REPEATED PLATE LOAD TESTS

Layer	Plate Diameter (in.)	Pressure Intensities (psi)
Subgrade	18	1, 2, 4, 8, 12
	24	1, 2, 3, 5, 8
	30	0.5, 1, 2, 3, 5
Pavements	8	10, 20, 40, 70, 100
	12	10, 20, 30, 40, 60
	18	5, 10, 15, 20, 30

Some aspects of the design, construction, and calibration of the stress and strain gages are given in the Appendix, and complete details are given by Wang, Mitchell, and Monismith (19). The stress gage was made of aluminum alloy casing with a foil type of strain gage cemented on the inner face of a diaphragm. Performance of the stress gages in pavement 1 was very satisfactory. The gages in pavement 2, however, did not perform as well, probably because the outputs of the gages in this pavement were too small to be read accurately.

The strain gage was composed of two $1\frac{1}{2}$ by $^3/_4$ by $^1/_8$ in. aluminum end plates used as reference points and a single linear variable differential transformer (LVDT) for measuring change in spacing between them.

The 0.001-in. dial gages used to measure deflections were attached to a 3 in. by 7 in. by 20 ft reference beam that was, in turn, anchored at points 10 ft away from the center of the test site and was stiffened laterally to prevent sway. For measuring deflection of the subgrade at a point under the center of the loading plate, a $^7/_{16}$-in. diameter hole was drilled to a depth of 1.5 in. above the top of the subgrade. A $^1/_8$-in. adjustable vertical rod was fastened to the bottom of the hole with Type III portland cement

With the aid of this test set up and instrumentation, it was possible to measure (as a function of plate size, applied pressure, and curing time for each pavement) the following quantities after any desired number of load repetitions:

1. Vertical deflections at the surface,
2. Vertical stress at the bottom of the stabilized layer (top of subgrade),
3. Radial strain at the bottom of the stabilized layer, and
4. Vertical deflection near the bottom of the stabilized layer.

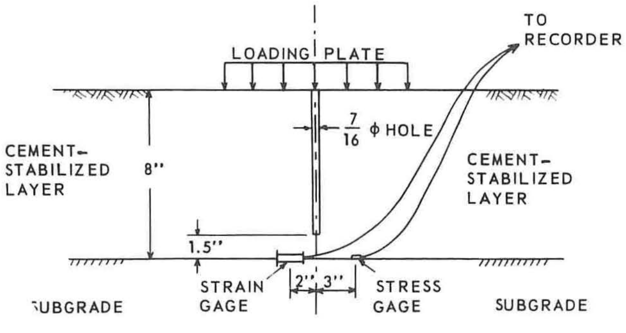

Figure 4. Schematic view of gage positions.

It was found that for any plate size, plate pressure, and curing time the stress and deflection responses were essentially constant after about 300 load repetitions. Thus, values reported subsequently are those corresponding to 300 load repetitions.

MATERIALS CHARACTERIZATION

In order to investigate the suitability of theory for prediction of stresses and deflections, certain material properties must be known or assumed for

each layer, e.g., strength, modulus, and Poisson's ratio. Both undisturbed specimens taken directly from the test pavement sections and specimens compacted in the laboratory were tested to provide this information. A summary of the test results is given in Table 3.

The unconfined compressive strength of the undisturbed specimens was about 50 percent of that of the laboratory-compacted specimens; the ratio of the resilient modulus in compression for undisturbed specimens to that for laboratory-compacted specimens was approximately 40 percent. The ratios for both flexural strength and resilient modulus in flexure are 69 percent and 57 percent respectively. It should be noted that the results for flexural tests were measured from only 4 undisturbed beam specimens. Difficulty in sampling precluded obtaining more beam specimens.

The large difference between strengths of field and laboratory-compacted samples were somewhat surprising because the field-mixing procedures were thought to be quite good. Nonetheless, the differences were significant, and the results point to a problem that remains largely unsolved when dealing with stabilized soils: how to predict properties of the stabilized layer in the field from the results of laboratory tests prior to construction. Because of these large differences, only the values obtained from the undisturbed specimens were used for the analyses described in the next section.

Although strength and modulus increased with increase in curing time and cement content for both compression and flexural loading conditions, the strain at failure under static load was constant regardless of sample age and cement content.

The resilient modulus in flexure was sensibly independent of the repeated stress level. The resilient modulus in compression, however, depended on both confining pressure and repeated stress according to the following expression:

$$M_R = K_1(K_2 - \log_e \sigma_d)I_1^{K_3} \tag{1}$$

where K_1, K_2, and K_3 are constants, σ_d is deviator stress, and I_1 is the first stress invariant.

A linear relationship between unconfined compressive strength and flexural strength was found; the modulus of rupture was about 20 to 35 percent of the unconfined compressive strength.

Although the modulus of resilient deformation in compression of the cement-stabilized soil could be represented by Eq. 1, the subgrade modulus of resilient deformation in compression was represented by an idealized bilinear function as shown in Figure 5 and expressed in the following form:

$$M_R = K_1 + (K_2 - \sigma_d)K_3, \text{ for } \sigma_d < K_2 \tag{2}$$

or

$$M_R = K_1 + (\sigma_d - K_2)K_4, \text{ for } \sigma_d > K_2 \tag{3}$$

TABLE 3

STRENGTH AND RESILIENT MODULI FOR UNDISTURBED AND LABORATORY-COMPACTED SPECIMENS

Specimens	Cement (percent)	Strengths (psi)		Resilient Moduli (10^3 psi)	
		Unconfined Compressive	Flexural	Compression	Flexure
Laboratory-compacted	3	60 - 110	15 - 40	40 - 150	60 - 180
	6	110 - 250	55 - 95	160 - 320	130 - 440
Undisturbed	3	20 - 50	—	10 - 95	—
	6	60 - 150	65	20 - 170	250
Ratio of undisturbed to laboratory-compacted	3	0.33 - 0.45	—	0.25 - 0.63	—
	6	0.55 - 0.60	0.69	0.13 - 0.53	0.57

Note: Ranges in values reflect influences of variations in curing period, density, and moisture content.

Development of these equations is given in an earlier report (19).

A summary of the values of properties and coefficients derived from the test results for the different materials and different curing times that are needed for analysis of stresses and deflections is given in Table 4. The lower values of subgrade modulus associated with the longer curing periods given in Table 4 resulted from an increase in subgrade moisture content. Poisson's ratio for the subgrade material was taken as 0.50 for analysis by elastic theory, as 0.48 for finite-element analysis, and as 0.20 for the cement-stabilized soil (3). (0.50 would be a more correct value, but the finite-element program cannot handle a value of 0.50.)

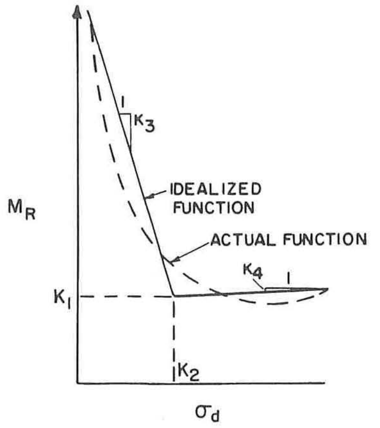

Figure 5. Characterization of subgrade modulus.

PREDICTION OF STRESSES AND DEFORMATIONS

Layered-elastic theory and finite-element analyses were used to predict surface deflections, radial strains at the base of the cement-stabilized layer, and vertical compressive stress on the top of subgrade near the centerline of the loading plate system for various curing times. The computed values were compared with the actual values measured in the repeated plate load tests.

Elastic-Layer Theory

Because the modulus of resilient deformation depends on the stresses, and the stresses and strains themselves are determined by the moduli values, a compatible solution requires successive approximations. Because the vertical and horizontal stresses induced in the prototype pavements by repeated plate loads may vary appreciably both vertically and horizontally, the resilient modulus within the pavement may vary considerably from place to place. Modulus variation in the vertical direction can be approximated by subdividing the pavement into several horizontal layers, each having a constant modulus throughout its thickness. Variations along a horizontal plane cannot be taken into account by using elastic-layer theory.

The behavior of the prototype pavements was analyzed by using successive approximations and an n-layer digital computer program developed by the Chevron Research Company (20). The pavement system was divided into 5 horizontal layers for analysis,

TABLE 4

PROPERTY VALUES USED FOR ANALYSES

Material	Curing Time (days)	Constants of Resilient Modulus in Compression (psi)				Resilient Modulus in Tension (10^3 psi)	Poisson's Ratio	Lateral Earth Pressure Coefficient
		K_1	K_2	K_3	K_4			
Soil stabilized with 3 percent cement	2	3,550	5.39	0.16	—	30.0	0.20	0.50
	23	3,550	4.52	0.44	—	45.0		
	100	3,550	4.12	0.65	—	60.0		
Soil stabilized with 6 percent cement	4	408	4.90	1.00	—	105.0		
	17	636	4.94	0.95	—	160.0		
	93	1,372	5.07	0.82	—	210.0		
Subgrade soils	2, 4, 23	14,500	4.5	1,000	-67.0	—	0.48	
	17	12,500	4.2	1,270	-100.0	—		
	93	3,800	2.4	2,250	-90.0	—		
	100	7,000	3.0	1,930	-85.0	—		

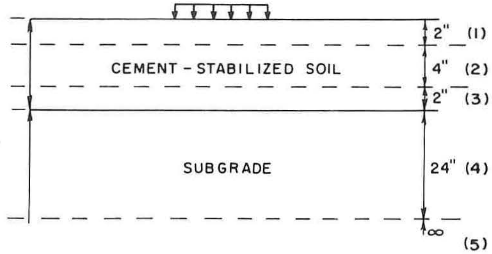

Figure 6. Five-layer representation of pavement section for analysis by elastic theory.

as shown in Figure 6, including 3 layers of 2-, 4-, and 2-in. thickness in the cement-stabilized soil section and 2 layers of 24-in. and infinite thickness in the subgrade. The surface load was assumed to be a uniformly distributed pressure on a flexible circular plate. The lateral earth pressure coefficient was assumed to be 0.50 throughout the entire pavement.

The procedure used for analysis was as follows:

1. Both the vertical and horizontal stresses at the mid-depth of each layer on the centerline of the loading plate system due to the surface load were estimated by using an approximate stress distribution (Boussinesq was used here);
2. The stresses induced by the weight of the pavement were added to the stresses estimated in step 1;
3. The approximate modulus of each layer in compression was estimated corresponding to the estimated stresses by using Eqs. 1 and 2, and the resilient modulus in flexure for tensile radial stresses was as given in Table 4;
4. The vertical stress and radial stress at the top and bottom of each layer were calculated by using the moduli determined in step 3, and the stresses at mid-depth of each layer were approximated by averaging;
5. The moduli corresponding to the stresses determined in step 4 were computed for each layer;
6. The moduli obtained in step 5 were compared with those estimated in step 3, and, if they were different, the whole procedure was repeated until the computed moduli were the same as the input values; and
7. When the estimated and computed moduli were in agreement, the deflections, strains, and stresses in the pavement were computed.

Finite-Element Analysis

The finite-element method of analysis has been proved to be an effective means for the analysis of axisymmetric solids by Clough and Rashid (6) and Wilson (21) and of pavement structures by Duncan et al. (7). In this method of analysis, the pavement structure is first idealized as an assemblage of a finite number of discrete structural elements interconnected at a finite number of joints or nodal points. The size of the elements are chosen to vary in accordance with the anticipated stress gradients. The finite elements for a pavement structure are actually complete rings in the horizontal direction, and the nodal points are in reality circular lines in plane view.

In the program used for this study, the pavement was divided into a series of quadrilaterals. Each quadrilateral was subsequently divided into 4 triangles by the computer program. Displacements were assumed to vary linearly within each triangle. The surface load was assumed to be a rigid circular plate load and to be applied stepwise so that the nonlinear stress-strain behavior and the modulus stress dependency of the cement-stabilized soil could be included in the analysis. Therefore, the accuracy of the solution is a function of the number of load increments used, with greater accuracy associated with smaller load increments.

The bottom boundary on which the nodal points are fixed was taken at a distance of about 50 radii beneath the pavement surface, and a vertical boundary on which the nodal points are constrained from moving radially was chosen at a distance of about 20 radii from the center of loading plate. A typical finite-element mesh for an 8-in. diameter loading plate is shown in Figure 7.

An element may be subjected to compression in one direction and tension in the other. For this case, the resilient modulus in compression was used only when the compressive strain was 5 times larger than the tensile strain because the ratio of failure strain

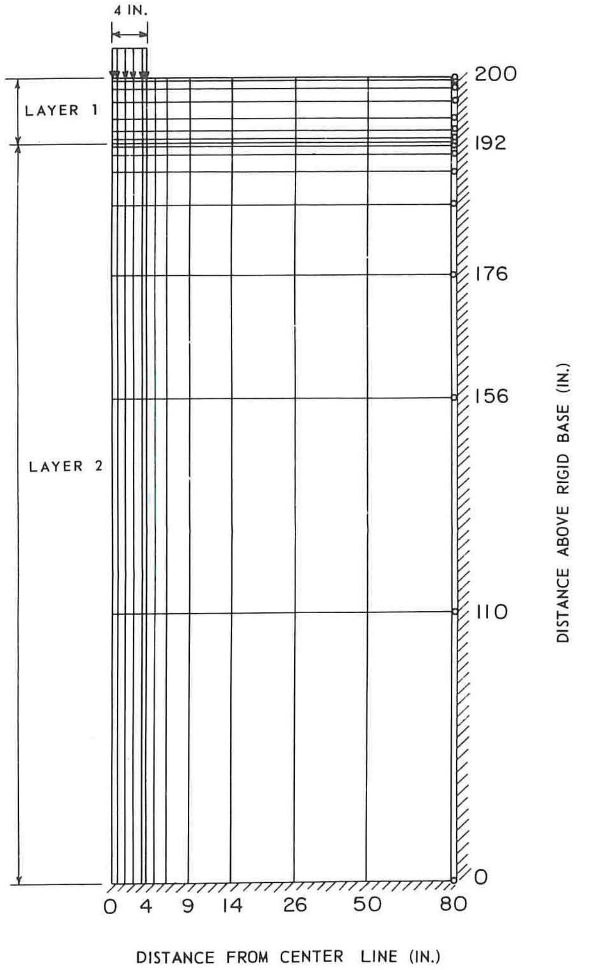

Figure 7. Finite-element configuration used for analysis of test pavements under 8-in. diameter plate.

in compression to that in tension equaled five. The element was considered to have failed when the shear strain induced from the applied load in the element reached the failure shear strain indicated by the laboratory test results. The resilient modulus of any failed element was assigned a small value (20 times less than the original value) for subsequent stress applications.

Resilient Deflections

Resilient deformations of pavement 1 cured for 2 days were predicted, and the results are shown in Figure 8. The resilient deformation of the pavement system agrees very well with the values computed by using elastic-layer theory. Shown in Figure 9 are the results of prediction of the resilient compressive strain in the cement-stabilized soil layer. It is seen that as the loading plate size increases the differences between predicted and measured values increase.

Although the finite-element analysis gives slightly lower predictions than the elastic-layer program, the agreement between the predicted and the actual deformations, especially at low plate pressures, is still quite good. Moreover, Figure 9 shows that the

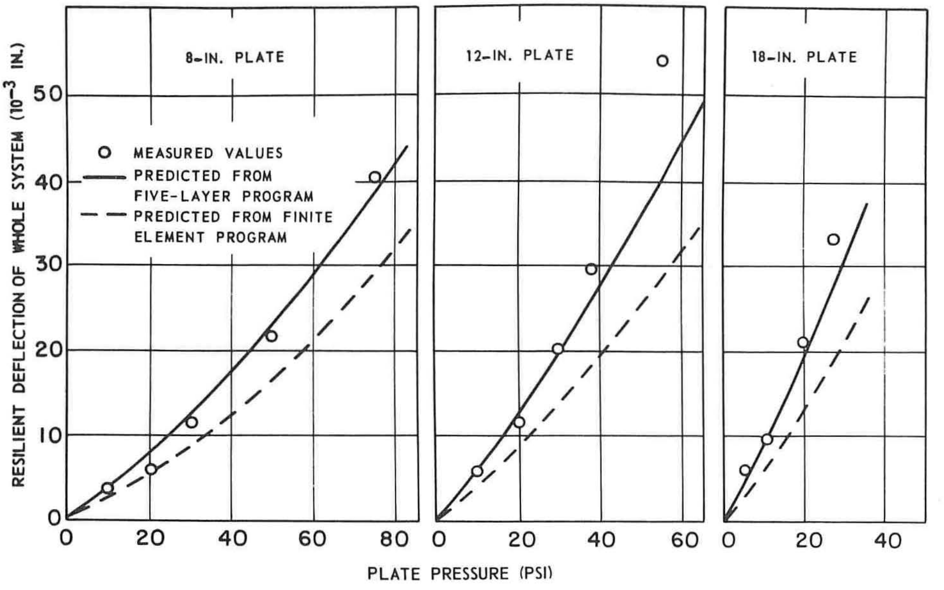

Figure 8. Resilient surface deflections in pavement 1 after 2-day curing.

finite-element analysis gives better predictions for the compressive strain in the stabilized layer than the elastic-layer program.

Figure 10 shows the predictions for surface deflection under a 12-in. diameter plate for pavement 1 after 23- and 100-day curing periods. The predictions, in general, are very satisfactory, and the analyses appear equally good for different curing times.

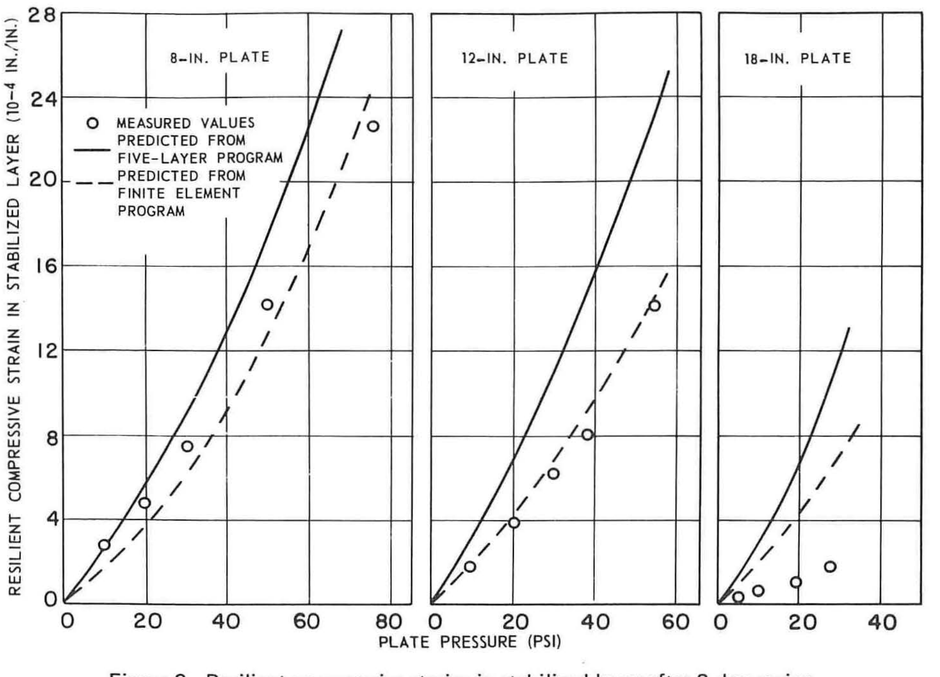

Figure 9. Resilient compressive strains in stabilized layer after 2-day curing.

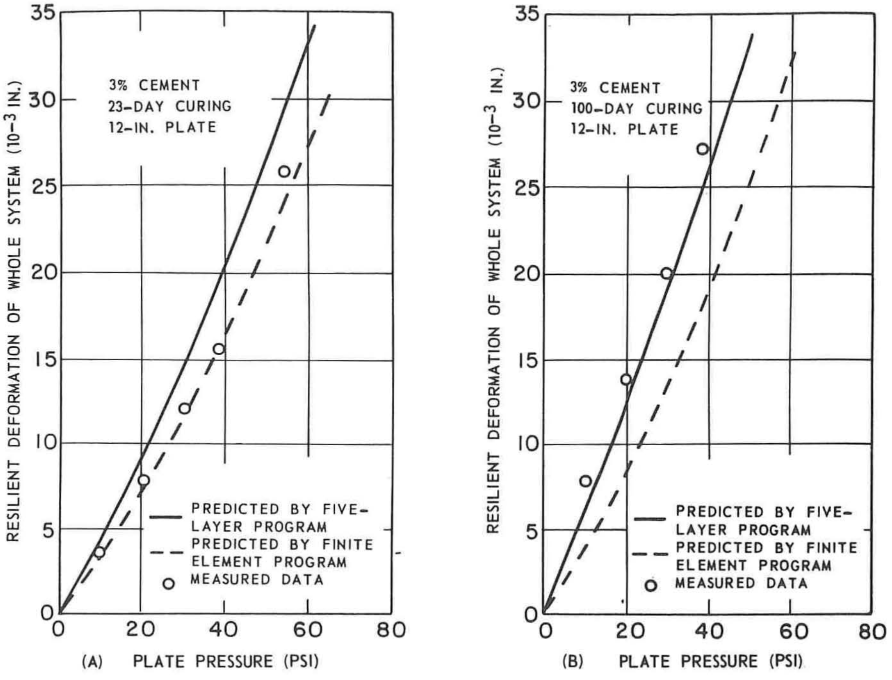

Figure 10. Resilient surface deflections in pavement 1 after 23-day and 100-day curing.

Figures 11 and 12 show the results of predictions for surface deflection pavement 1 because both theories predict values that are too high for all curing times. The results obtained from the finite-element analysis are in much better agreement with observation than those from elastic-layer theory, particularly at low plate pressure.

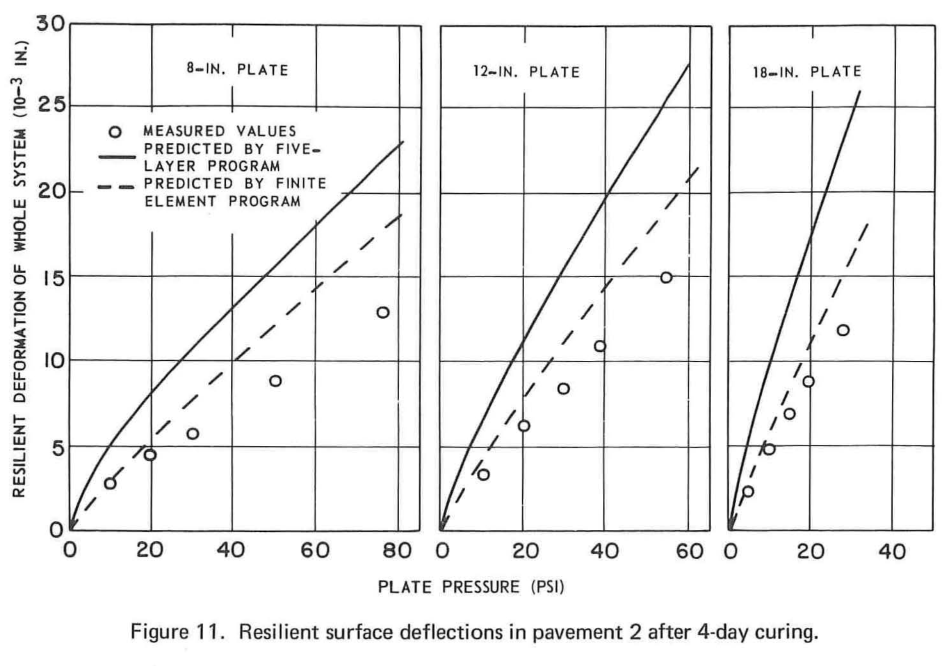

Figure 11. Resilient surface deflections in pavement 2 after 4-day curing.

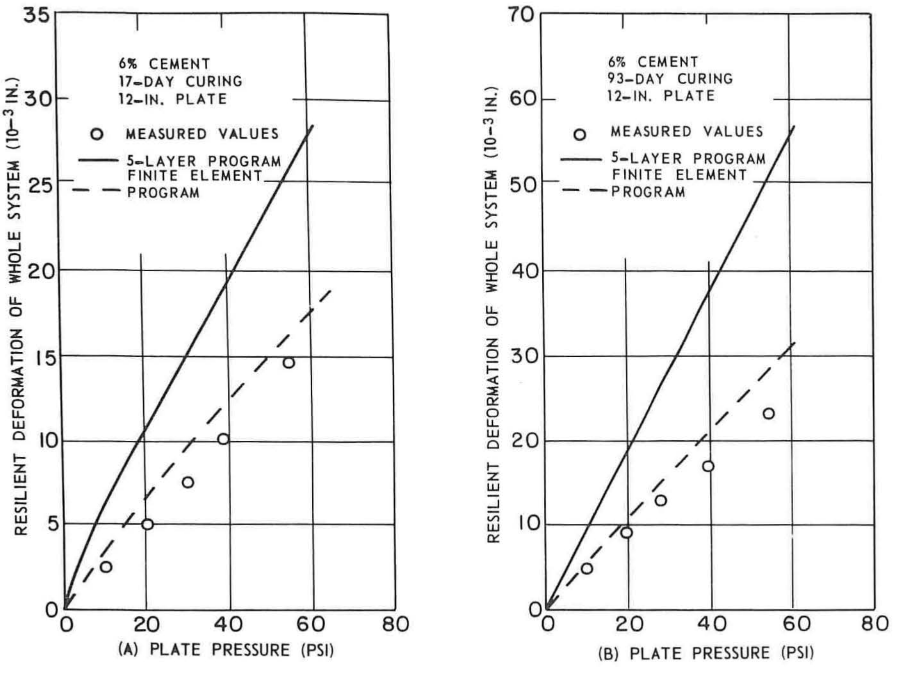

Figure 12. Resilient surface deflections in pavement 2 after 17-day and 93-day curing.

The results shown in Figures 8 through 12 indicate the following:

1. Elastic-layer theory gave better results than the finite-element analysis for the surface deflection in pavement 1. The finite-element analysis always underpredicted but still gave fairly reasonable results at low plate pressure.
2. For pavement 2, the finite-element analysis gave much better results than the elastic-layer theory but overpredicted somewhat.
3. Predictions were equally good at different curing times. This may imply that, although strength and modulus of the cement-stabilized soil increase with increase in curing time, the form of the stress-strain behavior did not vary significantly with time.
4. The finite-element analysis gave lower predictions than the elastic-layer theory; this behavior may be explained by the following. (a) The surface load was assumed to be a flexible circular load in the elastic-layer program, whereas it was assumed to be a rigid circular plate load in the finite-element program. Deflection under the rigid plate is constant throughout the whole contact area, whereas deflection under the flexible plate is a maximum at the center of the contact area. According to elastic theory for a homogeneous half space, the deflection under a rigid circular plate will be about 20 percent less than that under the center of a flexible loading of the same intensity. The assumption made in the finite-element analysis is more representative of the field test plate deflections than the assumption made in the elastic-layer program for the flexible loaded area. (b) In the elastic-layer program, the modulus of a layer is represented by the modulus corresponding to the stress acting at the intersection of the centerline of the loading plate and the mid-depth of the layer. Because the deviator stress on a horizontal layer is a maximum on the plate centerline, for a flexible surface loading, the corresponding modulus will be smaller than elsewhere. However, in the finite-element program, different moduli were used for different elements according to the state of stress acting on the elements. Therefore, the moduli values used in the elastic-layer program generally were less than those used in the finite-element program, leading to greater predicted deflections by the elastic-layer program than by the finite-element program.

Resilient Radial Strain at Bottom of Cement-Stabilized Soil Layer

Figures 13 and 14 show the results of predictions of resilient radial strain at the bottom of the stabilized layer for pavement 1 using both the elastic-layer program and the finite-element program. Both methods predicted very well at low plate pressure; but, as the plate pressure increased, the predicted values became smaller than the measured values. For longer curing times, both approaches gave better predictions at high plate pressure than at low plate pressure, and the finite-element program gave better predictions than the elastic-layer program.

The predictions made for pavement 2 are shown in Figures 15 and 16. The finite-element program predicted the values associated with a 4-day curing period very well. The elastic-layer program predicted slightly too high for all 3 plate sizes. For increased curing time, the predictions made from the finite-element program became poorer; on the other hand, the elastic-layer program predictions improved. The measured values are bracketed between the 2 predictions.

Vertical Compressive Stress on Top of Subgrade

The curves shown in Figure 17 compare measured and predicted values of vertical compressive stress on the top of the subgrade for pavement 1 after 2-day curing. Both the elastic-layer and the finite-element methods predicted well. However, as curing time increased, predicted values became larger than the measured values (Fig. 18). For longer curing times, the finite-element program gave better agreement than the elastic-layer program, but values were still overpredicted.

Both Figures 17 and 18 show that the finite-element program gave lower predictions than the elastic-layer program. The difference gradually decreased with increase in plate diameter. The reason for this may be as noted previously: Different assumptions for the surface load distribution were made in the elastic-layer program (assumed to be a flexible load) and in the finite-element program (assumed to be a rigid plate load). These different assumptions result in a difference in the distribution of contact pressure between the bottom of loading plate and the soil. The contact pressure under the flexible circular plate is constant over the loaded area, whereas the contact pressure

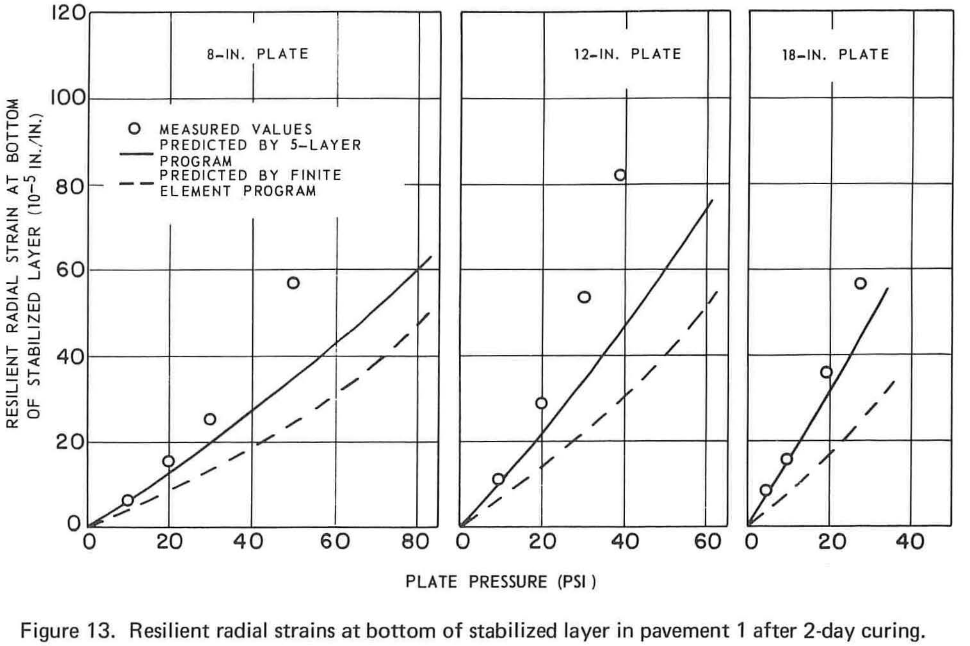

Figure 13. Resilient radial strains at bottom of stabilized layer in pavement 1 after 2-day curing.

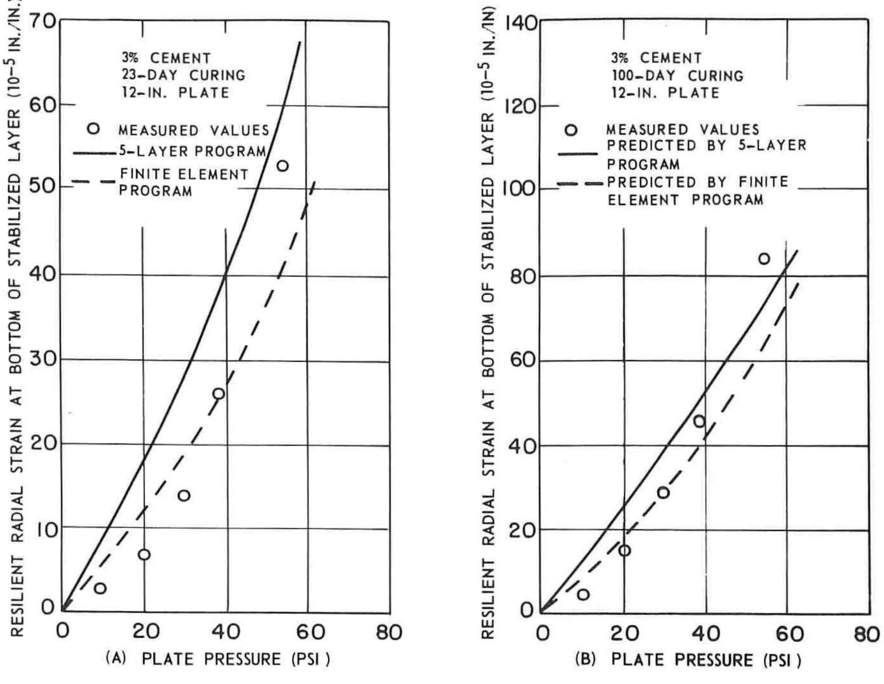

Figure 14. Resilient radial strains at bottom of stabilized layer in pavement 1 after 23-day and 100-day curing.

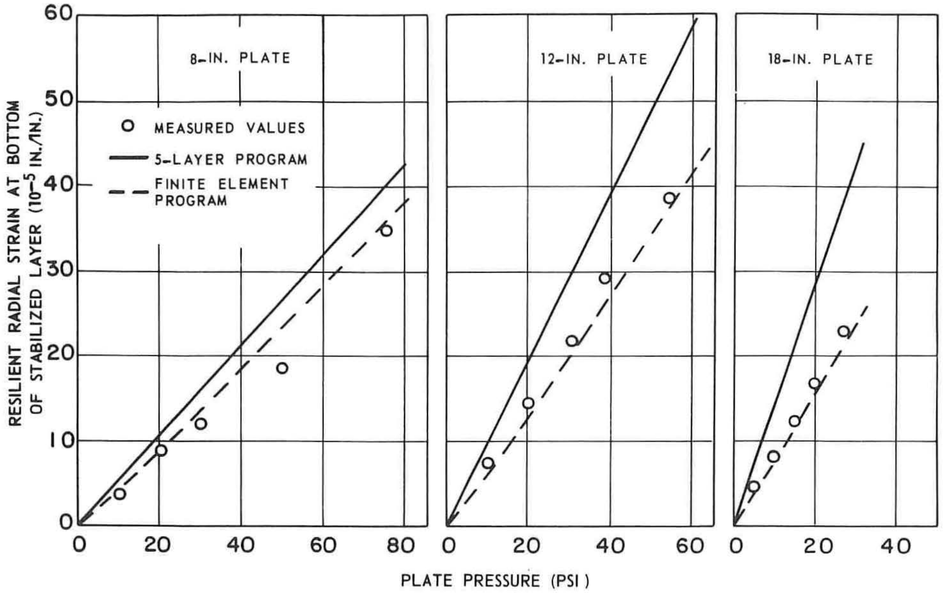

Figure 15. Resilient radial strains at bottom of stabilized layer in pavement 2 after 4-day curing.

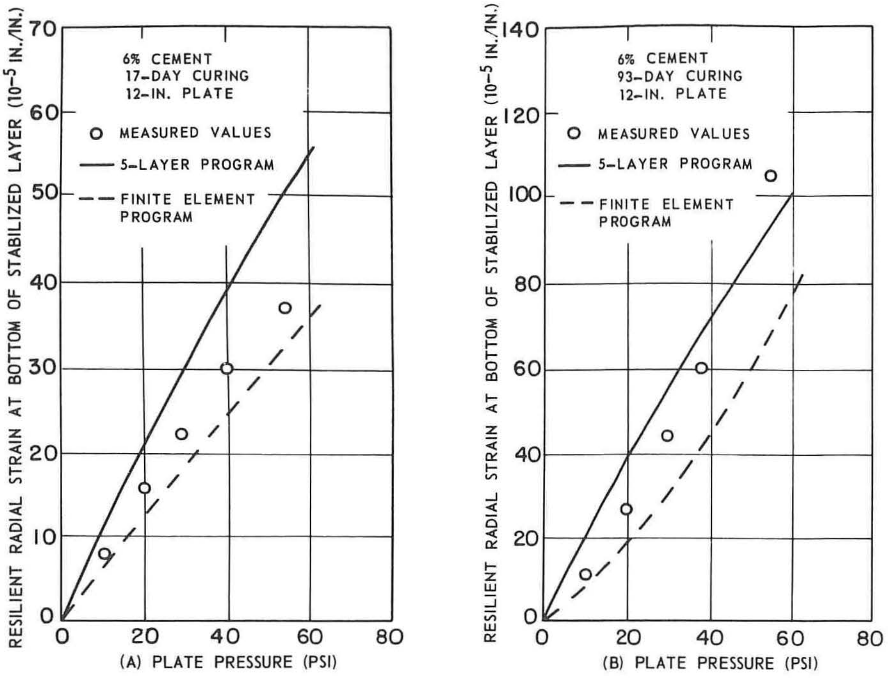

Figure 16. Resilient radial strains at bottom of stabilized layer in pavement 2 after 17-day and 93-day curing.

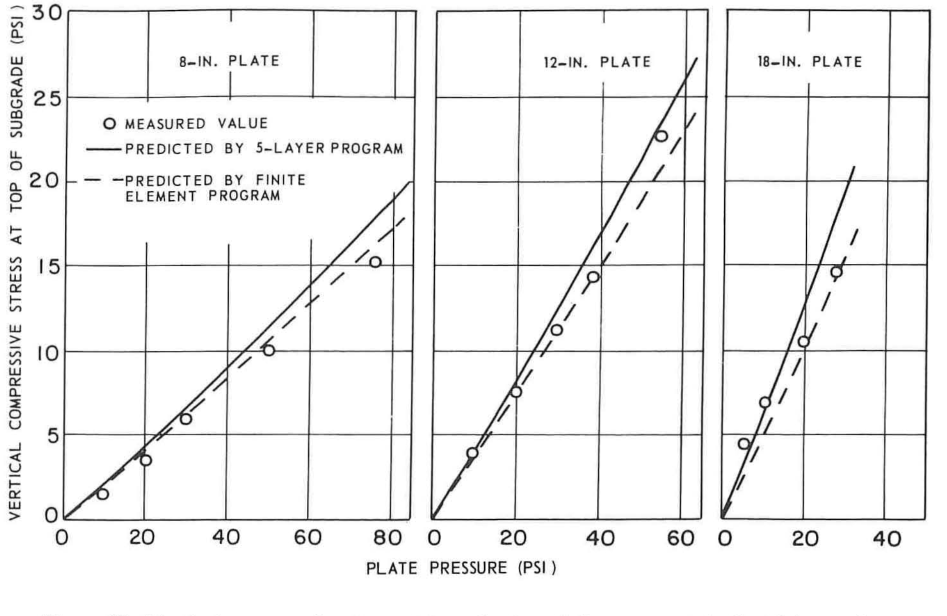

Figure 17. Vertical compressive stress at top of subgrade in pavement 1 after 2-day curing.

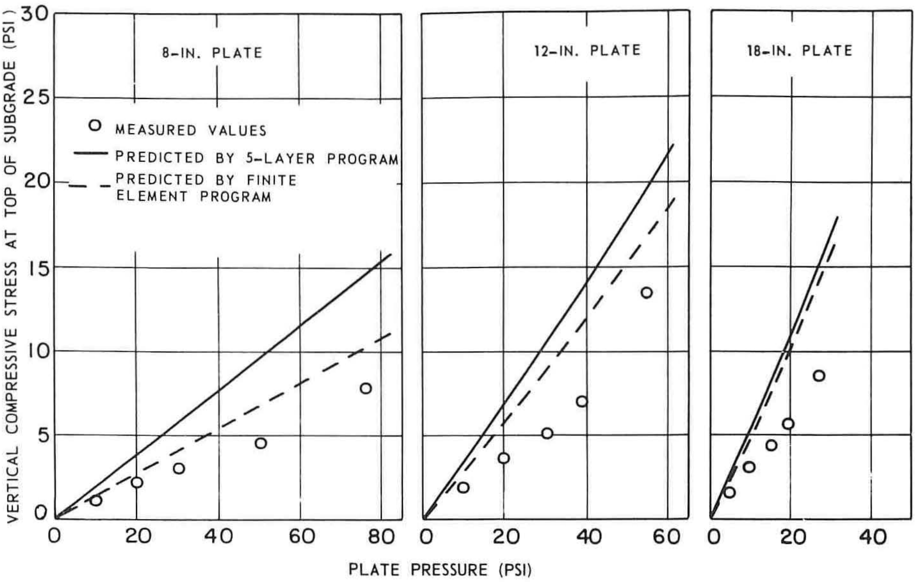

Figure 18. Vertical compressive stress at top of subgrade in pavement 1 after 23-day curing.

under a rigid plate is a maximum at the edges and a minimum at the center of the plate.

SUMMARY AND CONCLUSIONS

The primary purpose of this study was to investigate the applicability of available theories for predicting the induced stress and deflection response of 2 cement-stabilized soil pavement test sections to load. Both pavements consisted of a treated silty-clay soil layer, 8 in. thick overlying a clay subgrade. Pavement 1 contained 3 percent cement, whereas pavement 2 contained 6 percent cement. The study was divided into 3 phases: (a) field repeated plate load testing on 2 test pavements; (b) laboratory study of representative specimens, including laboratory-compacted and undisturbed specimens taken from the field, to determine appropriate material parameters to be used for prediction of stresses and deflections; and (c) prediction of stresses and deflections using available theories in conjunction with the appropriate measured values of material properties. This paper has been concerned mainly with the last phase.

Elastic-layer theory was found to predict quite well (a) surface deflection in pavement 1, (b) radial strain at the underside of the stabilized layer in pavement 1 at early stages of curing for low plate pressure, (c) radial strain in pavement 2 at later stages of curing, and (d) vertical stress in pavement 1 at early stages of curing.

Finite-element analysis predicted all field behavior under investigation reasonably well, except for the surface deflections in pavement 1 and the radial strains in pavement 1 at later stages of curing. Finite-element analysis always gave predictions lower than those given by elastic-layer theory. This may be attributed to the different assumptions for the plate flexibility.

The results of this investigation demonstrate that stresses and strains in cement-stabilized soil pavements can be predicted successfully by using elastic-layer theory and finite-element analysis together with material properties determined from laboratory repeated load tests on undisturbed specimens taken from the test pavements. Thus, a basis for pavement thickness design may be possible that limits critical stresses and strains within the pavement to acceptable values by using an approach similar to that suggested by Mitchell and Shen (16).

ACKNOWLEDGMENTS

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APPENDIX

STRESS AND STRAIN GAGES USED IN TEST PAVEMENTS

Stress Gage

Because the performance of stress gages is directly influenced by soil-gage interaction and because the interaction is a complex function of such factors as the gage dimensions and the gage-to-soil stiffness ratio, it is not feasible to design a gage that can eliminate gage error completely. However, the errors can be minimized by considering the following factors:

1. The thickness-to-diameter ratio should be as small as possible;
2. The ratio of the sensitive area of the gage face to the total facial area should be less than 50 percent;
3. The overall gage stiffness should be as high as possible because the gage cannot be made to always match the soil stiffness (the higher the stiffness is, the better the linearity will be);
4. For measuring stress at a point, the gage size must be small but large enough to minimize effects of nonuniformity of soil texture;
5. For dynamic load purposes, the density of the stress gage must be as close as possible to that of soil to reduce the effect of inertial forces on the stress gage response;
6. Long-term temperature compensation is desirable; and
7. The gage must be waterproof.

For the present investigation, the casing of the gage was made of aluminum alloy 6061-T651, and the strain gage was a full bridge in foil type, catalog No. FAES-4-70-12S13, manufactured by the Baldwin-Lima-Hamilton Corporation. This foil gage was cemented on the diaphragm with BR-610 cement. Details are shown in Figures 19 and 20. The gages were coated with LPS to prevent the reaction of cement with the aluminum casing.

Performance of the stress gages in pavement 1 (3 percent cement) was quite satisfactory. The gages in pavement 2 (6 percent cement), however, did not perform so well, probably because the outputs of the gages in this pavement were too small to be read accurately, even though the diaphragm thickness used in the gages for this pavement was reduced from the 0.45 in. used in pavement 1 to 0.30 in. In pavement 2 the reaction between gage and soil was probably similar to a system composed of a rigid plate overlying a gage that in turn is seated on a soft subgrade soil. The pressure transmitted from the pavement surface in such a system simply pushes the whole gage downward.

All stress gages were first calibrated by using statically applied air pressures, and 2 gages were calibrated in soil having the same characteristics as those of the pavement by using both static and repeated loads. The calibration factors for gages not tested in soil were obtained by assuming that the ratio of calibration factor for static air pressure to that for soil pressure remains constant for all gages having diaphragms of the same thickness.

A typical calibration curve is shown in Figure 21 for both static and repeated load conditions. The result indicated that static and repeated loads gave almost identical calibrations. A check calibration was made for one gage after the completion of field tests. It was found that calibra-

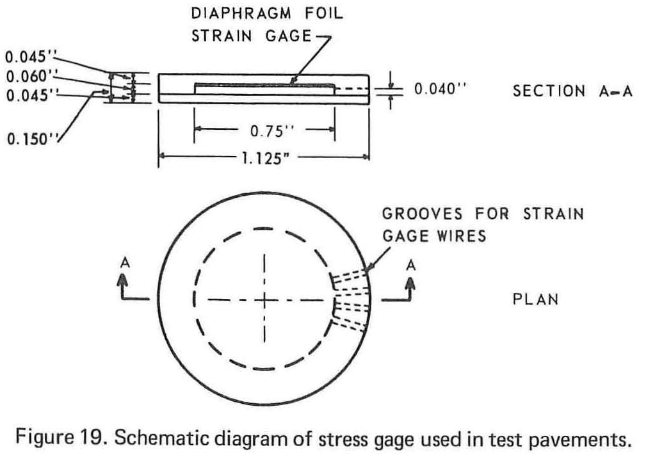

Figure 19. Schematic diagram of stress gage used in test pavements.

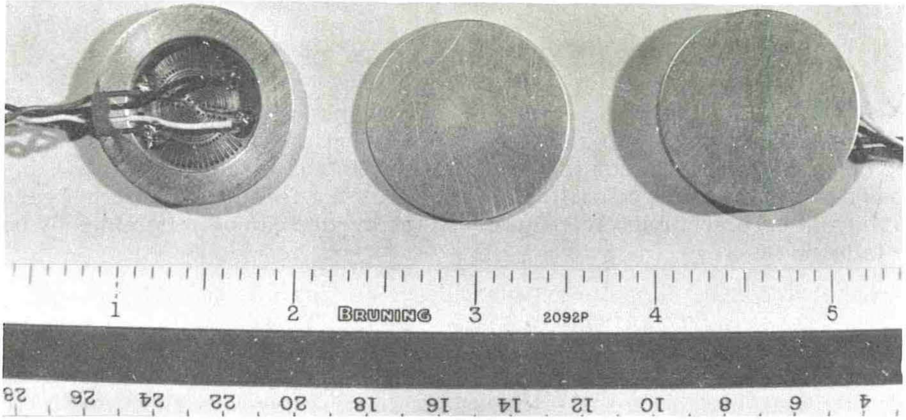

Figure 20. General view of stress gage used in test pavements.

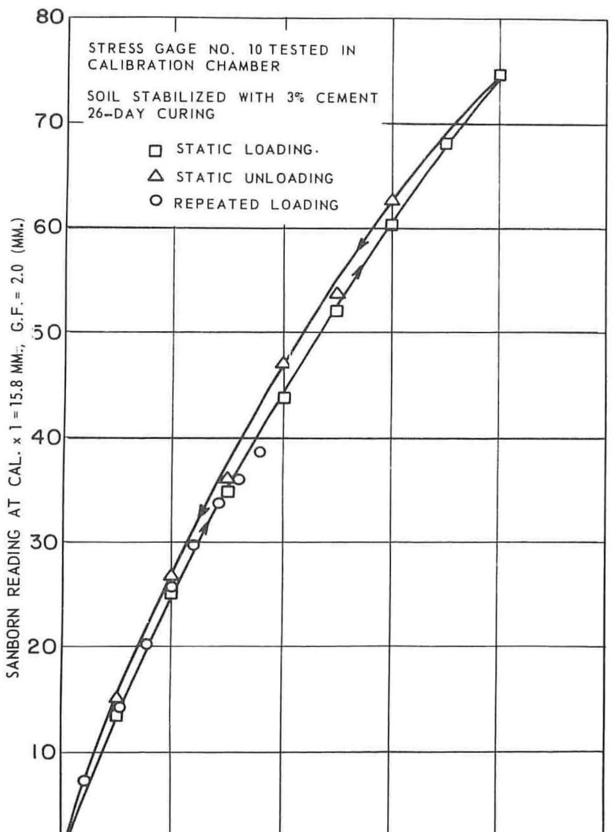

Figure 21. Typical stress gage calibration curve obtained from 20-in. diameter calibration chamber.

tion factor had not changed appreciably after almost 6 months of exposure in the pavement.

Strain Gage

The strain gage developed for the present study was composed of two $1\frac{1}{2}$-in. by $\frac{3}{4}$-in. by $\frac{1}{8}$-in. aluminum end plates and a single linear variable differential transformer (LVDT) for measuring change in spacing between the plates. The LVDT's used were manufactured by the Sanborn Company, catalog No. 595DT-100. The transformer coil assembly was clamped to one end plate, and the transformer core was screwed on a brass rod that was then fastened to the other end plate. The brass rod had 2 flexible joints so that relative movement of the 2 end plates would not cause friction between the core and coil assembly. A section of $\frac{3}{8}$-in. by 2-in. long polyethylene tubing was used to separate the 2 end plates and to envelop the LVDT to prevent intrusion of soil. Details and a general view of the gage are shown in Figures 22 and 23. All indications were that this type of gage performed very satisfactorily for the measurement of radial strains at the bottom of the stabilized layer.

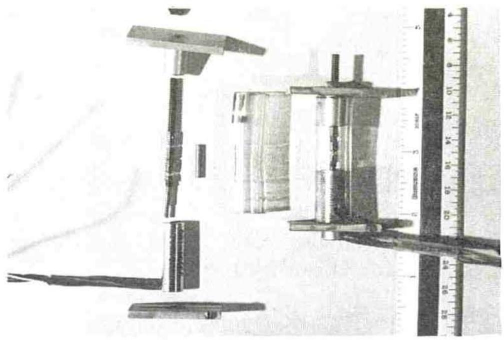

Figure 22. General view of strain gage used in test pavements.

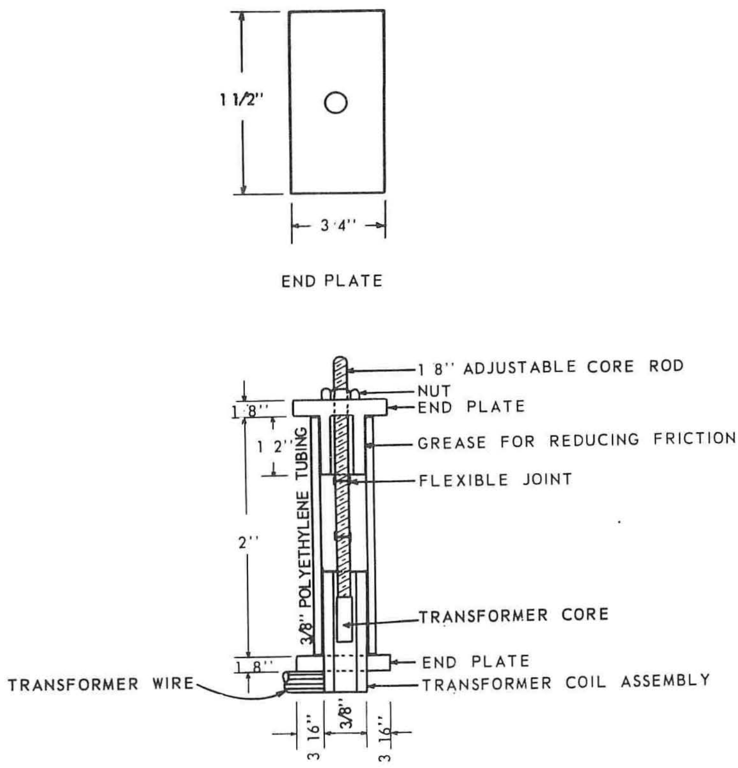

Figure 23. Detailed diagram of strain gage used in test pavements.

ESTIMATIONS OF INDIRECT TENSILE STRENGTHS FOR CEMENT-TREATED MATERIALS

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Center for Highway Research, University of Texas at Austin

•THE IMPORTANCE of the tensile characteristics of subbases can be demonstrated from both theoretical considerations and field observations. Nevertheless, until recently, little attention was given to the tensile characteristics of stabilized materials; thus, little information was available. Therefore, the Center for Highway Research at the University of Texas at Austin began a study to evaluate the tensile properties of stabilized subbase materials for use in pavement design, utilizing the indirect tensile or split-cylinder test (1). As a part of this study, a design procedure based on layered theory was developed.

To evaluate this pavement design procedure, performance data must be obtained from in-service pavements. Fortunately, various test sections exist in Texas for which performance data are available; however, there is no information concerning the tensile characteristics of the various materials used in their construction. Therefore, an attempt was made to develop correlations between indirect tensile strength and the results of both the cohesiometer test and the unconfined compression test, which are and have been used by the Texas Highway Department to evaluate cement-treated materials. The primary purpose for these correlations was to provide a means of estimating the tensile strengths of cement-treated subbases used in rigid pavements currently in service in order that the performance data collected during the life of the pavement can be used in the development of the subbase design procedure without the neccessity to wait for test sections to be designed, approved, funded, and constructed.

EXPERIMENT PROCEDURES

Two different correlation experiments were conducted. The first experiment was general in nature, and the second involved fixed curing and compaction conditions as specified by the Texas Highway Department. Thirty specimens were tested for each of these 2 correlations: 10 specimens in indirect tension, 10 specimens in the cohesiometer, and 10 specimens in unconfined compression.

The indirect tensile test specimens had a diameter of 6 in. and a height of 2 in. and were tested at 75 F at a loading rate of 2 in./min (1, 2, 3). The unconfined compression and cohesiometer tests were conducted according to the Texas Highway Department procedures (4). The unconfined compression test specimens had a diameter of 6 in. and a height of 8 in.; the cohesiometer specimens had a diameter of 6 in. and a height of 2 in.

Five factors were allowed to vary in the 2 correlation analyses. Three factors and their levels were the same for both correlations, and 2 factors were constant for a given correlation but were different for the 2 separate correlations. The 3 factors that were the same for both correlations and their levels were as follows:

Factor	Low	Medium	High
Molding water content, percent by weight	4	6.5	9
Cement content, percent by weight	2	6	10
Aggregate type	Gravel	—	Crushed limestone

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The constant factors were as follows:

Factor	General	Texas Highway Department
Aggregate gradation	Medium	Medium (well graded)
Curing temperature, deg F	100	75
Compactive effort	175 psi	25 blows/layer

Two different aggregates that are used extensively in central Texas were included in both correlation experiments. The first was a rounded gravel obtained near Seguin, Texas, which was relatively nonporous, and the second was a crushed limestone exhibiting high porosity. These 2 aggregates were separated and recombined to produce a well-graded mixture with a maximum aggregate size of $^7/_8$ in. and approximately 10 percent passing a No. 200 sieve. In addition to the 2 aggregate types, molding water content and cement content were allowed to vary at 3 levels.

The 2 remaining factors, which were associated with compaction and curing conditions, differed for each correlation. In the general correlation, all specimens were compacted by the Texas Gyratory shear compactor (3) at a compactive effort specified in terms of a compaction procedure and were then cured for 7 days at 100 F while wrapped in a PVC film. The specimens in the second correlation were compacted and cured according to procedures specified by the Texas Highway Department for cohesiometer and unconfined compressive test specimens. The specimens were compacted by using a Rainhart impact compactor, striking 25 blows per layer with a 10-lb hammer dropping 18 in. The specimens were then cured 7 days at 75 F in an environment of 100 percent relative humidity.

DISCUSSION OF RESULTS

Two correlation relationships relating the indirect tensile strength with the unconfined compressive strengths and the cohesiometer values were obtained for both the general conditions and for the conditions involving the Texas Highway Department curing and compaction procedures. In addition, a combined correlation analysis was conducted for all specimens. These 6 correlations, given in Table 1, were judged to be acceptable for the purposes outlined earlier. The combined correlation results are shown graphically in Figures 1 and 2.

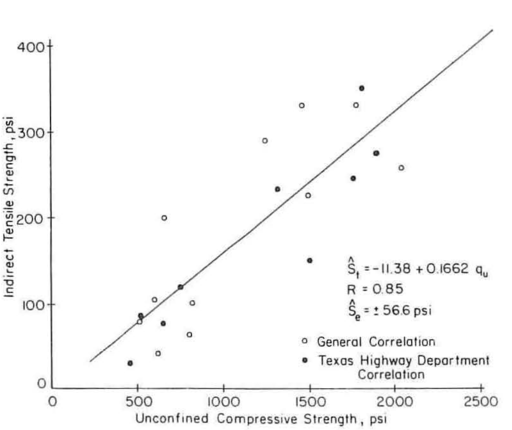

Figure 1. Indirect tensile strength and unconfined compressive strength relationship for combined correlation data.

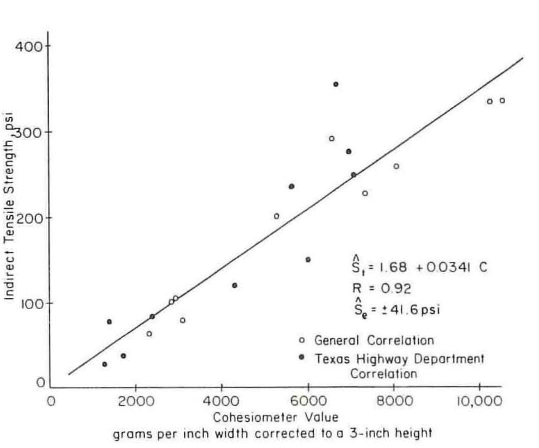

Figure 2. Indirect tensile strength and cohesiometer value relationship for combined correlation data.

TABLE 1

SUMMARY OF CORRELATION RESULTS

Correlation Variable	Equation[a]	Multiple Correlation Coefficient	Standard Error of Estimate (psi)	Coefficient of Variation (percent)
Indirect tensile strength versus unconfined compressive strength				
General	$S_t = 18.45 + 0.1548q_u$	0.80	67.2	34.3
Texas Highway Department	$S_t = -38.34 + 0.1752q_u$	0.93	43.8	27.3
Combined	$S_t = -11.38 + 0.1662q_u$	0.85	56.6	31.8
Indirect tensile strength versus cohesiometer value				
General	$S_t = 4.85 + 0.03200C$	0.96	30.4	14.5
Texas Highway Department	$S_t = -16.14 + 0.0403C$	0.91	49.3	30.8
Combined	$S_t = 1.68 + 0.0341C$	0.92	41.6	23.4

[a] S_t = predicted value of indirect tensile strength, psi; q_u = measured value of unconfined compressive strength, psi; and C = measured cohesiometer value, grams/in. of width corrected to a 3-in. height.

SUMMARY AND CONCLUSIONS

This paper presents a set of correlations from which indirect tensile strengths may be estimated from unconfined compressive strengths and from cohesiometer values (Table 1). It was found that correlations exist for these tests and that tensile strengths may be estimated from cohesiometer and unconfined compressive strength data. Nevertheless, these correlation relationships exhibited relatively high standard errors and coefficients of variation. Therefore, anyone attempting to use these relationship must judge their acceptability in terms of the needed accuracy and permissible error.

ACKNOWLEDGMENTS

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SHRINKAGE CRACKING OF SOIL-CEMENT BASE: THEORETICAL AND MODEL STUDIES

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This paper explores the phenomenon of shrinkage-induced cracking in soil-cement bases. An attempt was made to delineate the primary cause of cracking, and expressions for the shrinkage stresses were derived in accordance with linear viscoelastic theory. The results show that tensile shrinkage stresses are highly localized on the exposed surface. They attain maximum value during the first few days of drying and then decrease rapidly. For quantitative evaluation of shrinkage cracking, however, experiments were conducted on models whose design is based on a dimensional analysis of the linear problem of shrinkage cracking. Experimental results indicate that the crack intensity (defined as area of cracks per unit area) decreases with (a) an increase in the thickness of the slab and (b) a decrease in the viscosity of the material. Adequate extended curing is extremely effective in controlling cracking in cement base. In the 2 coarse-grained and 1 fine-grained soils studied, the crack intensity decreased when the cement content exceeded the ASTM-PCA freeze-thaw criterion. Also, crack intensity tended to decrease with an increase in subgrade friction. In a soil-cement matrix, relatively large pieces of gravel (nominal size ½ to 1¼ in.) enhanced cracking. The model was used in a search for treatments that led to several promising additives: lime and lime with a trace amount of sugar proved to be best in a variety of soils; expansive cement admixture and sodium silicate surface treatment are effective in coarse-grained soils.

•THE WIDESPREAD phenomenon of cracking in stabilized pavements is generally considered to be caused by a combination of drying shrinkage and ambient temperature during and after the curing period. The fact that cracks may develop in a new pavement before the application of any wheel load indicates the importance of stress caused by changes in temperature and moisture content. Westergaard (32) considered 2 conditions to be involved in this problem. The first arises from slow, uniform shrinkage; the second arises from quick changes of temperature (or drying of a slab from the top) occurring, for example, by the change from a hot day to a cool night and vice versa. Later studies (15, 26, 30) have extended Westergaard's theory to include the effects of a simple harmonic temperature variation, partial support, nonlinear temperature distribution, and viscous damping forces. An obvious conclusion of all these investigations is that warping can induce stresses of sufficient magnitude to cause cracking of concrete highway pavements.

The mechanics of cracks in soil-cement base, or any pavement component for that matter, have not been sufficiently investigated by engineers and researchers. The few studies (19, 25) conducted in this area simply consider the intensity of visible cracking on the pavement surface. As yet, the basic cause of the problem has not been extensively studied. George (7), in a recent paper, presented simplified solutions to crack-spacing and crack-width problems. In that study, the assumption was made that the

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material is perfectly elastic. Some implications of a more realistic assumption—that the material is viscoelastic—are discussed in a second paper (10); and in yet another, the cracking problem is examined in accordance with the theory of brittle fracture advanced by Griffith (12). In the present paper, the cracking problem is studied by employing models whose design is based on a dimensional analysis of the linear problem of shrinkage cracking.

The primary objectives are to investigate the mechanics of shrinkage cracking and to delineate the factors influencing the cracking of stabilized soil-cement bases. Whereas some qualitative results are obtained by a theoretical analysis, for quantitative information this investigation utilizes the techniques of dimensional analysis and small-scale model experiments. These models are employed in a further objective of investigating the effectiveness of additives in reducing the shrinkage cracks in soil-cement base.

STRESS ANALYSIS OF PAVEMENT BASE SUBJECTED TO RESTRAINED SHRINKAGE

An investigation of the shrinkage-induced cracking of pavement base entails the following steps: (a) stress analysis of the slab bonded to the subgrade and shrunk by drying from the top face and (b) determination of a suitable failure criterion. A failure criterion based on ultimate tensile strength is reasonably practical (16) and, hence, will not be discussed here. The first step mentioned, however, is regarded to be so complex a problem that a review of the existing experimental and analytical solutions can be enlightening.

Stress Arising From Base-Subgrade Interaction

In a completely constrained pavement slab, a uniform shrinkage will give rise to uniform stress throughout the depth of the slab without any distortion. If the slab is only partially constrained, however, or is subjected to nonlinear strain variation throughout the depth, it will tend to warp. In many instances, warping stresses can be sufficiently high to cause cracking of the slab.

Stresses Due to Warping

If a pavement slab is subjected to a temperature gradient, or to a moisture gradient that results from drying from the top face, its surface will tend to warp. For a linear strain distribution, Westergaard (32) solved the problem for a slab of finite width and infinite length, supported on a Winkler foundation. Considering that warping due to nonlinear temperature variation may result in only partial support of the slab by the ground and assuming that subgrade reactions are time-dependent, Reddy, Leonards, and Harr (26) presented expressions for stresses and deflections in a finite circular slab.

Stresses Due to Restraint

When a material expands or contracts uniformly but is not allowed to do so at some parts of its boundaries, a phenomenon of restrained shrinkage occurs. In the pavement-cracking problem, the underlying subgrade offers the restraint to the shrinking base. So far, solutions for only 2 limiting cases of restraint are available (4, 31). They are reviewed in the next section.

Stress in a Slab When Bonded on One Face to a Rigid Plate and Shrunk—Assume a hypothetical case of a plate perfectly bonded to a foundation and shrunk. Let S be the free shrinkage (that is, $\epsilon_{xx} = \epsilon_{yy} = S$, for $0 \leq z \leq h$) in the plate. For the Cartesian system shown in Figure 1, the boundary conditions at the bottom face are

$$u = Sx;\ v = Sy;\ \text{and}\ w = 0 \tag{1}$$

which result in

$$\epsilon_{xx} = \epsilon_{yy} = S \tag{2}$$

where

u, v, w = displacements in x-, y-, and z-directions; and
$\epsilon_{xx}, \epsilon_{yy}$ = unit elongations in x- and y-directions.

Using Hooke's law and Eq. 2, we can express the stresses at the interface by

$$\sigma_{xx} = \sigma_{yy} = ES/(1 - \nu) \tag{3}$$

where

E = modulus of elasticity of the slab material; and
ν = Poisson's ratio of the slab material.

Although the theoretical determination of stresses in the top of the plate is complex, Durelli (4) has solved this problem by employing photoelastic models. The stress obtained at the center of a 14-in. by 14-in. by 1-in. epoxy slab is

$$\sigma_{xx} = \sigma_{yy} \simeq 2\ ES \tag{4}$$

where σ_{xx}, σ_{yy} = normal components of stress parallel to x- and y-axis.

Stress in a Free Slab—Contrary to conditions found in a plate bonded to a rigid foundation, in an unrestrained plate the stress due to uniform shrinkage will be zero. If the shrinkage is linear but is not symmetrical with respect to the central plane, the stresses can be computed by the following expression (31):

$$\sigma_{xx} = \sigma_{yy} = -\frac{ES_z}{1 - \nu} + \frac{1}{h(1 - \nu)}\int_0^h ES_z dz + \frac{12\ z}{h^3(1 - \nu)}\int_0^h Es_x zdz \tag{5}$$

where h = thickness of slab.

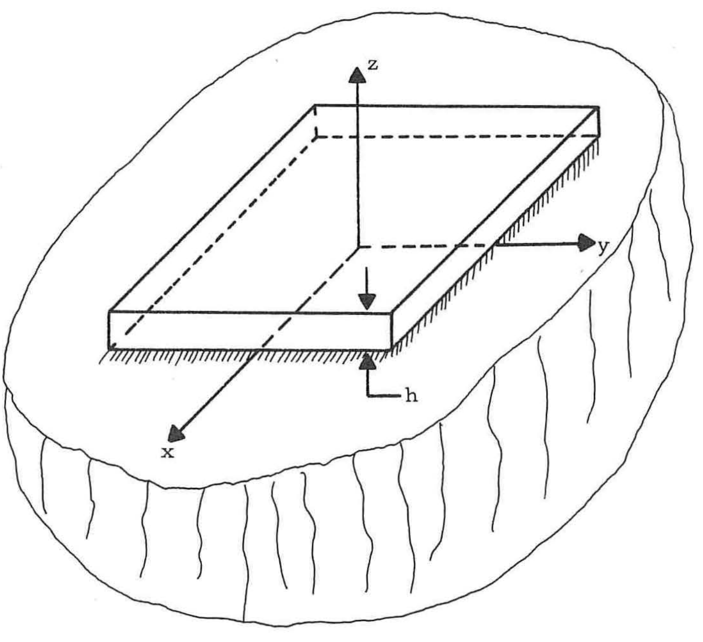

Figure 1. Slab bonded on one face and shrunk.

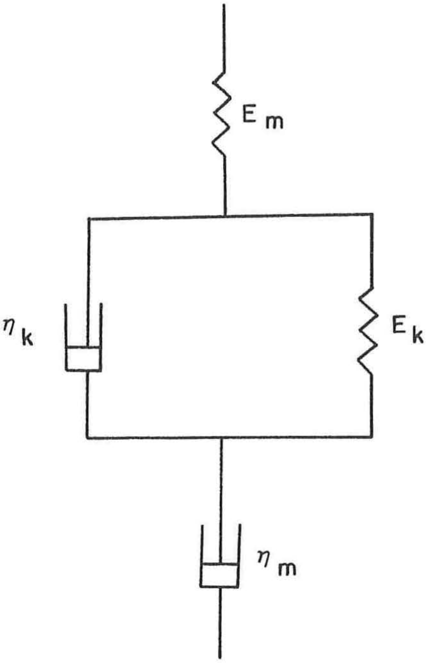

Figure 2. Rheological (Burgers) model of soil-cement for soil K03 at 6 percent cement ($E_m = 0.89 \times 10^6$ lb/in.^{2}; $\eta_m = 4.50 \times 10^7$ lb-hr/in.^{2}; $E_k = 1.70 \times 10^6$ lb/in.^{2}; and $n_k = 1.89 \times 10^7$ lb-hr/in.^{2}).

This discussion reveals that the stress level in a shrinking slab is primarily dependent on 2 factors: shrinkage strain distribution and subgrade restraint. The latter will be shown to depend on the former. Because both of these factors vary as the drying shrinkage progresses, a rigorous analysis of the stresses in a shrinking pavement slab can be complex. Given in the following, however, is a simplified analysis of the shrinkage stresses in a soil-cement slab permitted to dry from the top face only.

Theoretical Analysis of Shrinkage Stress

Before expressions for shrinkage stress can be derived, assumptions must be made in regard to laws controlling the flow of moisture in soil-cement and the relationship between shrinkage and moisture loss as well as the stress-strain-time relation. Assuming that the weight flux of water perpendicular to the direction of flow is directly proportional to the potential gradient and that the movement of water obeys the principle of conservation of matter, Gardner (6) obtained the 1-dimensional flow equation

$$\frac{\partial \theta}{\partial t} = D_{(\theta)} \frac{\partial^2 \theta}{\partial z^2} \tag{6}$$

where

θ = moisture content, percent; and
$D_{(\theta)}$ = diffusivity, in.^{2}/hour.

The diffusivity in soil-cement, unlike that in natural soils, can be assumed to be nearly constant over a limited range of moisture change. Equation 6, with constant diffusivity, has been solved (2); a solution specifically applicable to the present problem is given by Sanan and George (27) and by Pickett (23). The shrinkage versus moisture content (weight loss) relationship has been experimentally determined (8) and found to be approximately linear except during the final stages of drying. Burgers model, as shown in Figure 2, is chosen to describe the stress-strain-time relation of soil-cement (10). These assumptions were used in computing shrinkage strain, S, in a slab drying from the top face (27), which is shown in Figure 3a. These strain distributions, at various indicated times, are exact solutions for a slab infinite in the x-y plane. These curves indicate that during the early stages of drying the exposed surface is subjected to severe shrinkage strains.

The next step in the solution is to consider the base subgrade interaction problem. The theoretical model chosen for analysis employs the bending of an infinitely long slab (y-direction) of width B (x-direction) resting on homogeneous foundations whose reaction against the slab is proportional to the deflection (Winkler foundation). As a result of drying, the slab is subjected to shrinkage strain that varies nonlinearly with depth (Fig. 3a) but remains constant on any given plane parallel to the surface of the slab.

The expressions for the stresses in the x- and y-directions are given in another paper (27). Stresses on the exposed surface have been calculated by employing these equations and are plotted with time in Figure 4. The tensile shrinkage stresses attain maximum value during the early stages of drying (40 to 100 hours, depending on restraint condition) and then decrease comparatively rapidly. The maximum shrinkage stress varies from $0.208\ E_m S_\infty$ with no restraint to $0.273\ E_m S_\infty$ with complete restraint. Theoretically, regardless of the restraint condition, shrinkage stress is highly localized on the exposed surface and decreases sharply with depth (Figs. 3b and 3c). This highly localized stress can be relieved only by surface cracks or by plastic flow in the material.

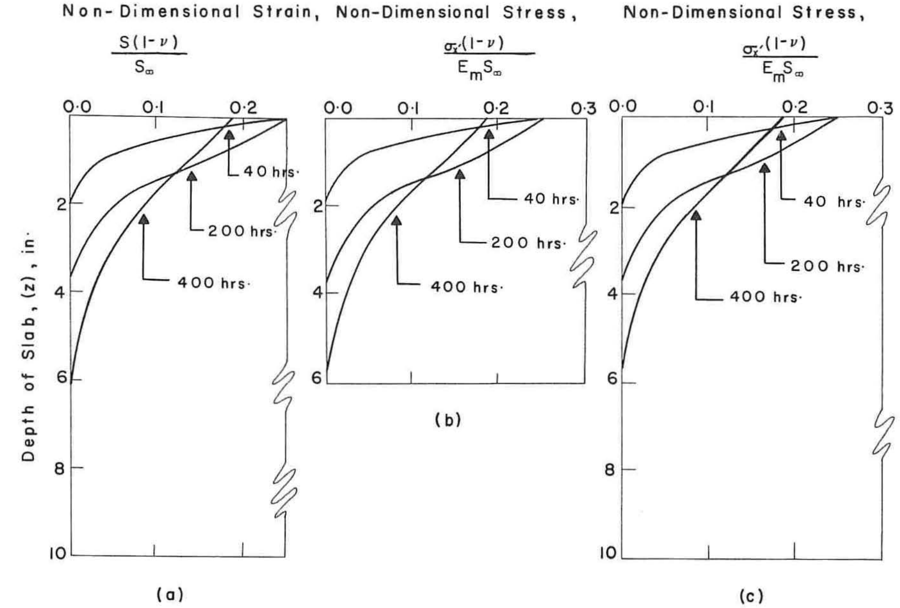

Figure 3. Shrinkage stress and strain distributions in a restrained slab at various times for soil K03-6.

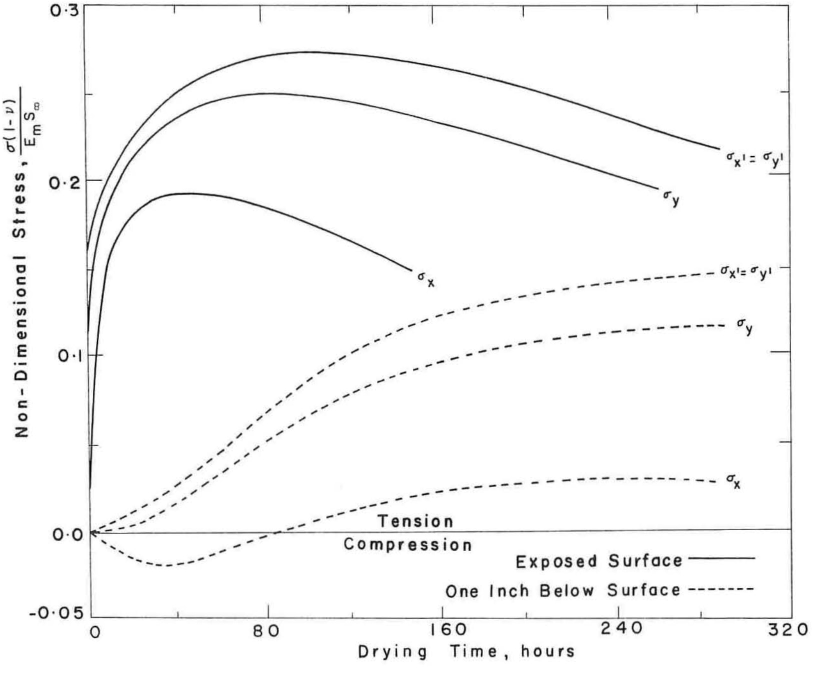

Figure 4. Variation of tensile shrinkage stress (viscoelastic) with drying time for slab 8 in. deep and for soil K03-6 (σ_{x1}, σ_{y1} = stress for complete restraint, and σ_x, σ_y = stress for no restraint).

If yielding takes place, there will be redistribution of stresses on the surface, a condition that has not been accounted for in this analysis. It may be interesting, therefore, to examine the stress variation at some interior point close to the exposed surface. The graphs shown in Figure 4 (indicated by broken lines) indicate the stress distribution at a point 1 in. beneath the exposed surface. As expected, the stress intensity decreases by 50 to 80 percent, whereas the time to attain maximum stress increases to as long as 200 to 350 hours.

In summary, the linear viscoelastic theory predicts that the shrinkage stress in a pavement slab subject to drying from one face is highly localized on the exposed surface and decreases sharply with depth. This highly localized stress can be relieved either by surface cracks or by plastic flow in the material. The limitations of the stress analysis presented should, therefore, be recognized because the 4-element model, especially at failure, does not necessarily represent the exact stress-strain-time behavior of soil cement. Furthermore, the stresses, to an important degree, are dependent on the subgrade restraint. In pavement slabs, therefore, the complex interactions among many factors involved during drying preclude a general study of the cracking until a number of gaps in the knowledge concerning these factors have been closed. For quantitative results, therefore, this investigation utilizes the techniques of dimensional analysis and scale-model experiments as described in the following section.

SCALE-MODEL STUDIES

A model is a device that is so related to a physical system that observations on the model may be used to predict the performance of the physical system in the desired respect. Kondner (17) has previously demonstrated the effectiveness of this approach in the field of soil mechanics. Although pavement cracking is not amenable to study by a "true" model, it is highly desirable to proceed with a somewhat distorted model study.

Formulation of the Problem

The design of the experimental model is based on a dimensional analysis of the linear problem of shrinkage cracking. The principal advantage of dimensional analysis lies in reducing the number of variables that must be investigated and in formulating advantageous dimensionless variables.

The extent of cracking in a stabilized soil-cement base, or any pavement for that matter, can be denoted by the area of cracks, I. The important factors that affect cracking are the geometrical dimensions of the base, the stress-strain-time characteristics of the material and its tensile strength, the resistance offered by subgrade, and the rate and extent of shrinkage. The geometry of the base may be defined by its length L, breadth B, and thickness H. The base is usually cast continuously; a construction joint, however, may occur every 80 to 100 ft.

If the material can be idealized by a Maxwell fluid, the stress-strain characteristics of the base may be described by the Young's modulus E and modulus of viscosity η. As postulated earlier, the cracking depends on the tensile strength of the material, σ_u, which is assumed to remain constant throughout the test.

Subgrade interaction is a complex process. Friberg (5) has stated that, because of temperature variations or other causes, movement in the slab occurs over a certain length (called the active length) from the free end. Over a major portion of the active length the frictional resistance is constant and depends on the weight of the pavement. The resisting force in this investigation, however, is assumed to be constant over the entire length and is taken to depend on the frictional resistance per unit area per unit depth, $F_\mu = \mu\gamma$.

In the field, a soil-cement base is subjected to the combined effects of drying shrinkage and fluctuations of temperature. The latter, of course, influence the rate of shrinkage. In this study, however, the models are exposed to an atmosphere of constant temperature and relative humidity (RH). Even then, shrinkage strain may be a nonlinear function of time as shown in Figure 5 (10). This condition can be approximated, however, by a bilinear curve as shown in the figure. The pertinent variables, therefore,

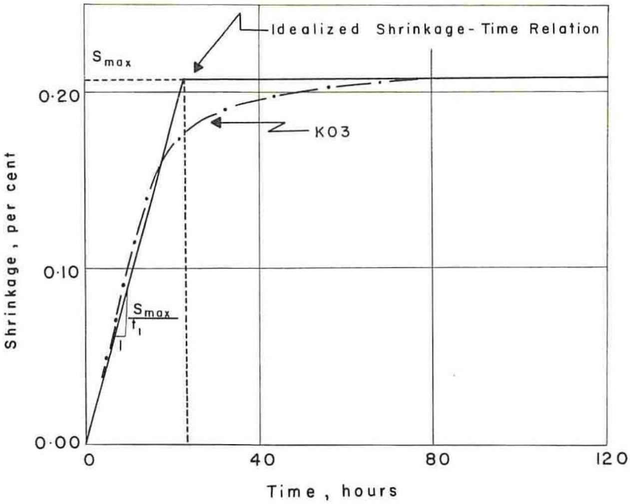

Figure 5. Typical time-rate of shrinkage of soil-cement when air-dried at 55 RH and 72 ± 4F.

may be taken as the rate of shrinkage, $\dot{S}_{av}$, and the time to attain maximum shrinkage, t_1. For the time being, it is assumed that the variation of shrinkage throughout the depth of the pavement does not cause any warping; what affects the phenomenon is the average shrinkage.

These considerations result in the set of physical quantities given in Table 1. There are 11 variables, and by examination it can be determined that rank of the dimensional matrix is 3. So there are 8 independent dimensionless products, which give the following functional relationship:

$$\frac{I}{LB} = f\left(\frac{B}{L}, \frac{H}{L}, \frac{E}{\sigma_u}, \frac{\eta}{HF_u t_1}, \frac{\eta}{\sigma_u t_1}, \dot{S}_{av} t_1, \frac{t}{t_1}\right) \tag{7}$$

If the material used in the model is the same as that in the prototype, by artificially treating the foundation to increase μ, the dimensionless product $\eta/(HF_\mu t_1)$ in the model may be made equal to that in the prototype. If a scale factor of $\frac{1}{10}$ is chosen as the geometrical scale, the coefficient of friction should be increased 10 times.

For a particular soil, S_{max} (= $\dot{S}_{av}t_1$) is constant and, therefore, can be omitted from the list. Tests have to be conducted separately, however, for each type of soil. When we alter the cement content, if we assume that E is increased in proportion to σ_u, the factor E/σ_u can be taken to be constant and therefore omitted from Eq. 7.

In practice the breadth of the pavement cannot be altered to control cracking. It must be held constant. The term H/L is different from B/L, however. Changing the thickness may affect t_1; for example, t_1 increases with the thickness. Also, F_μ may be affected by a change in thickness. Because H/L may not affect the crack intensity in a direct manner, it may be deleted from Eq. 7.

Having made the restriction that σ_μ and η be the same in the prototype and in the model, we can satisfy the equation $(\eta/\sigma_u t_1)_m = (\eta/\sigma_u t_1)_p$ by having $(t_1)_m = (t_1)_p$. This requirement was complied with by drying the experimental model in 55 percent RH. For the 3 soils tested, this requirement resulted in a drying period of approximately 15 days for the model that corresponded to the same period in the prototype. The assumption here is that under extreme field conditions the prototype pavement undergoes drying in approximately 15 days.

Using the same soil as in the prototype pavement, with the simplifications introduced and the restrictions imposed,

TABLE 1

PHYSICAL QUANTITIES CONSIDERED IN THE DIMENSIONAL ANALYSIS OF CRACKING OF SOIL BASE

Physical Quantity	Symbol	Dimension
Geometry		
Length of pavement	L	L
Breadth of pavement	B	L
Thickness of pavement	H	L
Variables controlling stress-strain-time characteristics		
Young's modulus	E	FL^{-2}
Modulus of viscosity	η	$FL^{-2}T$
Tensile strength	σ_u	FL^{-2}
Variable affecting subgrade resistance		
Resisting force/unit area/unit depth = μ_γ	F_μ	FL^{-3}
Variables affecting shrinkage		
Shrinkage rate	$\dot{S}_{av}$	T^{-3}
Time	t	T
Time to reach maximum shrinkage	t_1	T
Dependent variable		
Area of cracks	I	L^2

we can write the functional relation for a specific soil as

$$\frac{I}{LB} = f\left(\frac{\eta}{HF_\mu t_1}, \frac{\eta}{\sigma_u t_1}, \frac{t}{t_1}\right) \tag{8}$$

Experimental Procedure

Materials—Model tests were conducted in 3 soils, each at 3 to 4 different cement contents. Classification properties of these soils are given in Table 2. For convenience, each soil is identified by a1-letter, 4-digit system; for example, K03-06 means soil with 6 percent cement whose predominant clay mineral is kaolin. Type I portland cement was used in this study.

Model Testing—The model of the base consists of a 4-ft by 2-ft by 1-in. thick soil-cement slab with 1 of its short edges fixed and the other 3 edges free. Simulating the subgrade underneath is an aluminum plate whose interlocking (frictional) characteristics are modified to satisfy the similitude requirements. If a geometrical scale of $1/10$ is used, the model so designed simulates half of a prototype pavement slab of size 80 ft by 20 ft by 10 in. thick.

The soil-cement mixture, which has been mixed slightly above the optimum moisture, is compacted by a 45-lb roller to approximately 90 percent of the AASHO T-99 density. After casting, the slab is kept in a fog room (RH nearly 100 percent and temperature 72 ± 2 F) for 7 days. After curing in the fog room, the model is dried by exposure in 55 percent RH at 72 ± 2 F. The field condition is simulated by sealing all surfaces except the top face, and the slab is allowed to dry through the top face only. Cracks that appear as the slab dries eventually form a definite pattern in which orthogonal intersections predominate. The length and width of the cracks that occur as the slab dries are the important data collected from the model tests.

Rosette strain gages cemented to a corner at the free end of the model are used to measure shrinkage strain. The fact that the corner of the slab can be assumed to be stress free ensures that the creep strain and strain due to stress are both nearly zero. The shrinkage strain is thus obtained from the strain gage. (Shrinkage strain is that unit deformation due to any cause other than stress that would occur in an infinitesimal element if the element were unrestrained by neighboring elements.)

The investigation of the factors responsible for pavement cracking entails the evaluation of the crack intensity due to variation of the 3 dimensionless products on the right side of Eq. 8. Because the explicit relationship between the dependent variable and the other dimensionless products is not known, it is desirable to limit each parameter within a range consistent with the prototype pavement. The dependent parameters are varied as follows: (a) F_μ by changing the frictional characteristics of the aluminum plate, (b) σ_u by changing the cement content in the soil, (c) t by taking measurements at various intervals, and (d) t_1 by covering the surface so as to reduce the rate of drying.

When cement content of a soil is varied to change σ_u, both t_1 and η change and thus affect the dimensionless factor $\eta/(HF_\mu t_1)$ in Eq. 8. Experimental results, however,

TABLE 2

SOIL CLASSIFICATION DATA

Characteristic	Sand Clay (K03)	Sand Clay (K36)	Silty Clay (M30)
Liquid limit, percent	31	22	37
Plasticity index, percent	10	1	13
-2 micron clay, percent	16	16	20
Mineral composition of -2 micron clay	Kaolinite	Kaolinite	Montmorillonite and illite
Classification			
AASHO system	A-2-4	A-2-4	A-6-9
Unified system	SM	SM	ML
Textural system	Sandy	Sandy	Silty clay

show that η/t_1 remains nearly constant for any specific cement that ensures that Eq. 8 is the governing relation.

RESULTS AND DISCUSSION

The experimental data from the model tests are presented here. Several tests were repeated to ensure duplicability. The crack intensity, i(= I/LB), as defined in this report, is the unit of area of cracks per unit area of slab ($in.^2/in.^2$). In reporting the crack intensity, however, weight factors such as 1, 0.8, and 0.5 are assigned respectively to initiate, lengthen, and widen a crack. These factors are semi-empirical and have been chosen so that the factor associated with each of these events is inversely proportional to the probability of its occurrence. By modifying lengthening and widening in this manner, the author hoped to obtain the crack potential of the shrinking slab. The following example illustrates how these weight factors can be used to determine a crack intensity value in a model. The increase in crack intensity, Δi, due to an increase in width (weight factor 0.5) from 0.02 to 0.03 in. of a 1-in. long crack (per unit area) would be 0.005 $in.^2/in.^2$ Adjustment of the measured crack area as indicated, however, does not change the qualitative nature of the results presented in this section.

Graphical Representation of the Model Test Results

As indicated earlier, Eq. 8 governed the design of the model. With one exception, all of the geometric design conditions were satisfied in the model. It was not possible to reduce the size of the soil particles used in the laboratory to the same scale as the other geometrical parameters. This contradiction in geometrical similitude between the model and the prototype can be overcome by assigning an appropriate distortion factor for each soil type. Inasmuch as the principal objective of this investigation has been to delineate the factors affecting cracking, an exact numerical value for the distortion factor cannot be of great value; this discrepancy, however, precludes a direct comparison of the results of soils with different textures. The fact that a significant quantity such as S_{max} has been deleted from the general functional relation, Eq. 8, further invalidates the scope of combining the results of fine-grained and coarse-grained soils. Of necessity, therefore, the results of these 2 groups of soils will be analyzed separately. Because relatively more data are available for the latter group of soils, the discussion that follows will primarily concern granular soils.

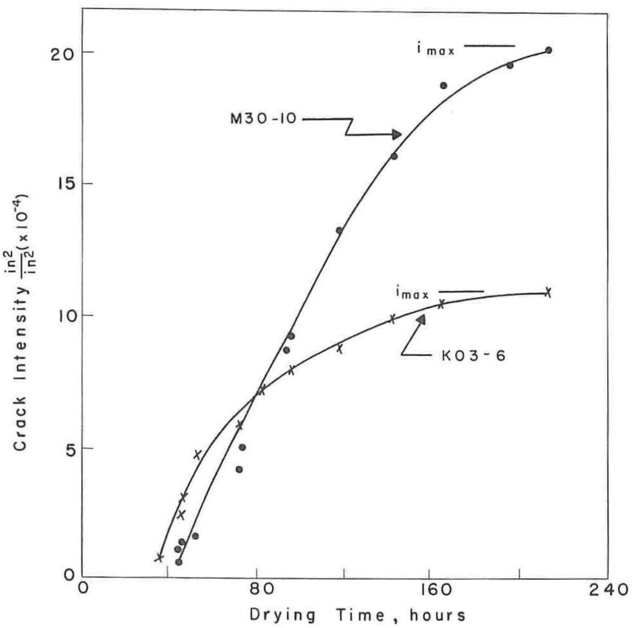

Figure 6. Typical curves of crack intensity versus drying time.

Typical plots showing the increase in crack intensity with time are shown in Figure 6. The maximum crack intensity, i_{max}, in each test has been estimated from similar plots, and its variations with the other dimensionless parameters, $\eta/(\sigma_u t_1)$ and $\eta/(HF_u t_1)$, are shown in Figure 7. Because the maximum crack intensity is of interest in this study, t/t_1 can be taken to be unity and, therefore, deleted from Eq. 8. The following conclusions from the plot seem warranted:

1. Crack intensity, i, increases with modulus of viscosity (η), and the use of such additives as

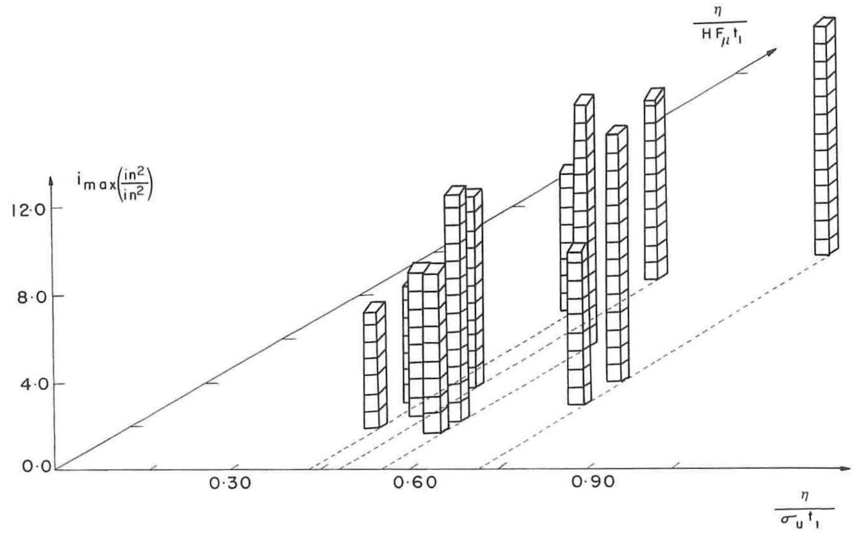

Figure 7. Crack intensity, i_{max}, related to 2 dimensionless parameters (crack intensity decreases with decrease in parameters).

emulsion or rubberized asphalt should be considered for decreasing viscosity and, therefore, minimizing cracks; and

2. Crack intensity decreases with an increase in thickness of the slab H, the time taken to attain the maximum shrinkage t_1, the tensile strength σ_u, and the coefficient of subgrade friction μ.

The importance of thickness can be demonstrated by a comparison of the theoretical stress distribution in the interior of slabs 6 in. and 10 in. thick. It can be observed in Figure 3b that, when the drying time of a 6-in. slab is increased from 40 to 400 hours, the critical depth (defined as the zone in which the shrinkage stress exceeds the strength of the material) increases from 20 percent of slab depth to 60 percent. When the same time interval is observed in a 10-in. slab, however, the critical depth increases from 10 percent to only 35 percent. In other words, because tensile stress exceeds the strength, a 6-in. slab will crack more in a given time than will a 10-in. slab.

When the slab is unrestrained, however, shrinkage stress can be shown to be a function of the characteristic of the system, λ, where

$$\lambda = (K/4D)^{1/4} \tag{9}$$

where

$D = Eh^3/[12(1 - \nu^2)]$; and
K = modulus of the foundation.

It has been verified (15, 27) that warping stresses and, thereby, shrinkage stresses in an unrestrained slab decrease with an increase in the flexural rigidity, EI, or a decrease in the modulus of the foundation. In other words, tensile stresses due to warping are lowered significantly more by thick slabs on weak subgrades than by thin slabs on strong subgrades.

The significance of increasing t_1, that is, the desirability of adequate curing, is discussed elsewhere (10). The findings of that study show conclusively that cracking in cement base can be minimized by adequately extended curing. The last 2 factors, namely σ_u and $F\mu$, have been varied in the present investigation; consequently, the results and the significance of these factors on cracking are discussed in some detail.

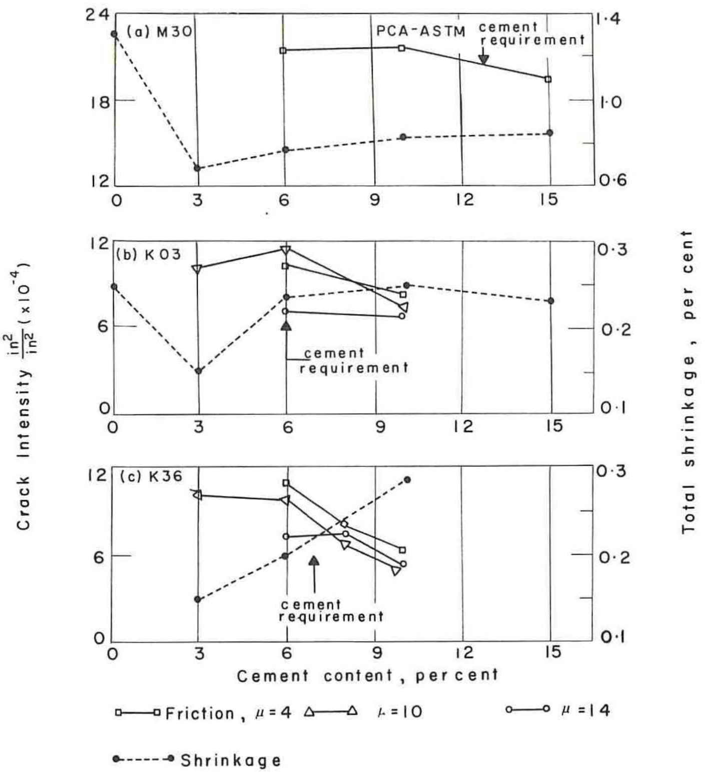

Figure 8. Variation of crack intensity with cement content in soils M30, K03, and K36 for various values of subgrade friction.

Effect of Cement Content (Tensile Strength) on Crack Intensity

In Figure 8, the 2 curves for granular soils are shown to have certain characteristics in common. That is, for treatment levels ranging from a small percentage of cement to that specified by the Portland Cement Association (PCA) criterion (according to ASTM D 560), the crack intensity remains nearly constant. Thereafter it begins to decrease. For granular soils, therefore, it may be desirable to use slightly more cement than is required to meet the PCA-ASTM criterion.

Although cracks result primarily from shrinkage, it is paradoxical that the crack intensity results do not agree with the shrinkage results reported in a previous study (9). Typically, as shown by broken lines in Figures 8b and 8c, the minimum shrinkage occurred in cement proportions somewhat below the PCA requirement; thereafter shrinkage increased as cement content was increased. At low cement contents, although shrinkage is at a minimum, the base, on account of its low tensile strength, tends to exhibit closely spaced cracks. Conversely, when cement content is increased, tensile strength is also increased and cracks occur at wider spacings. The increased crack spacing can be shown to have a twofold effect on crack width and, therefore, on crack intensity. First, as evidenced by the equation (7)

$$\delta_Y = S_\infty L - \frac{\mu \gamma L^2}{4E_t}$$

where

δ_Y = crack width,

S_∞ = maximum (final) shrinkage,
L = crack spacing,
μ = coefficient of subgrade friction,
γ = unit weight of base, and
E_t = Young's modulus in tension,

crack width decreases with L^2. Second, as shown by Pickett (22), the creep strain, which arises from the restraint in the slab, increases with slab size and partly compensates for the shrinkage, thereby reducing the crack width. In summary, the higher tensile strength overcomes the adverse effect of increased shrinkage.

The crack intensity in the fine-grained soil, M30, is nearly constant with the increase in cement content, a fact in satisfactory agreement with the shrinkage results (Fig. 8a). In fine-grained soils, a cement content that meets the PCA-ASTM criterion should give a satisfactory mix for design purposes.

Effect of Subgrade Friction on Crack Intensity

In the fine-grained soil, the crack intensity remains unchanged, whereas in the 2 coarse-grained soils, as shown in Figure 7, the crack intensity tends to decrease with μ. In explaining these results, it may be noted that crack intensity is influenced by 2 opposing factors. As μ increases, frictional resistance increases (7, Eq. 1a) creating frequent cracks. The data given in Table 3, which show that the crack length increases with increase in μ, support this hypothesis and perhaps suggest an increase in crack intensity as well. The increased restraint, however, opposes this tendency, as it enhances the creep in the shrinking matrix and partly compensates for the shrinkage to reduce the crack width and, thereby, crack intensity. The effect of subgrade will be even more significant in the field. That is, the high frictional resistance may serve to redistribute more evenly the stresses caused by localized shrinkage and, thereby, reduce the incidence of cracking. The theoretical studies of Pickett (22) as well as the experimental results obtained by Nagataki (20) show that as restraint increases the resultant strain assumes nearly uniform distribution.

The fact that crack intensity in the fine-grained soil model is not significantly influenced by the subgrade resistance can be the result of warping in cracked panels. As drying of the model slab continued, cracks appeared. After further drying at constant temperature, these cracked panels separated out of the base and exhibited large convexity at the top face (11). It was concluded, therefore, that crack intensity during the later stage of drying cannot be a function of the subgrade resistance.

Test results indicate that increasing the subgrade friction tends to reduce crack intensity. The observation that mixed in-place jobs exhibit less cracking than do central plant jobs validates this finding.

Cracking Influenced by Texture of Soil

By necessity, the gradation or texture of soil is not taken into consideration in designing the model. It does not seem feasible, therefore, to relate crack intensity to textural characteristics such as clay content. Indirectly, however, a relation can be demonstrated here. The effect of clay content on shrinkage has been reported (9), and the results show that the shrinkage increases somewhat exponentially with -2 micron clay content.

From the results of the present investigation, it can be shown that crack intensity is influenced by shrinkage. As given in Table 4, for example, the crack intensity increases somewhat proportionally with the drying shrinkage. It must be emphasized that, of all factors pertaining to soil, the clay exerts the most influence in cracking of soil-cement base.

TABLE 3

ULTIMATE CRACK LENGTH OBSERVED IN MODEL

Soil	Cement (percent)	Length of Cracks[a] (in.)		
		μ = 4	μ = 10	μ = 14
K03	3	79.5 (30.5)	91.0 (29.5)	
	6	63.0 (35.0)	84.0	
	10	25.0 (21.5)	35.0 (28.0)	34.5 (26.5)
K36	6	66.0	54.0	
	8	43.0	35.0	
	10	44.0	41.5	70.0
M30	6	103.5		113.0
	10			
	15	64.0		103.0

[a]Maximum shrinkage strain, $S_\infty \times 10^{-4}$, is in parentheses.

The effect of coarse aggregates in cracking is somewhat understood. By acting as rigid inclusions in the shrinking matrix, they reduce shrinkage (9, 24). The part played by the gravel, approximately ½ to 1¼ in. nominal size, often found in cement-treated clay-gravel mixtures, is discussed in the following.

A recent study (29) on the shrinkage microcracking in concrete reveals that tensile stresses, developing at the aggregate-cement paste interface, cause microcracks at the interface during the curing period of concrete. Several studies concerning the disturbing effect of small spherical and cylindrical inclusions on an otherwise uniform stress distribution are reported in the literature (3, 13, 28). In simple tension, for example, a perfectly rigid spherical inclusion intensifies the tension at the "poles," in the same direction as the applied tension T, to about 2T (13).

TABLE 4

CRACK INTENSITY COMPARED WITH MAXIMUM SHRINKAGE

Soil	Cement (percent)	Maximum Unit Shrinkage (percent)	Crack Intensity (in.²/in.² × 10^{-4})
K03	6[a]	0.2407	11.38
K36	6	0.2071	9.86
M30	10	0.7506	21.52

[a]Cement based on dry weight of soil, as a percentage.

Aggregates, therefore, assume a dual role in the shrinkage cracking of soil-cement: First, by acting as rigid inclusions in the shrinking matrix, they inhibit shrinkage; and, second, by virtue of the stress concentration at the interface, they tend to enhance cracking. This problem has been investigated by model testing.

Inasmuch as the model design followed the principles of dimensional analysis, it was desirable to proportion the aggregate size accordingly. It is shown elsewhere (13) that stresses in 2 systems (model and prototype), with inclusions of the same form but different sizes, are the same at similarly situated points. A requirement in the model design that the shrinkage stresses in the model and in the prototype be the same at homologous points makes it imperative that gravel of the same size be used in the 2 systems. Because of practical considerations, however, natural gravel of nominal size—½ to ¾ in.—is used in this investigation. The gravel was randomly embedded in the M30-10 matrix at distances of approximately 10 in.

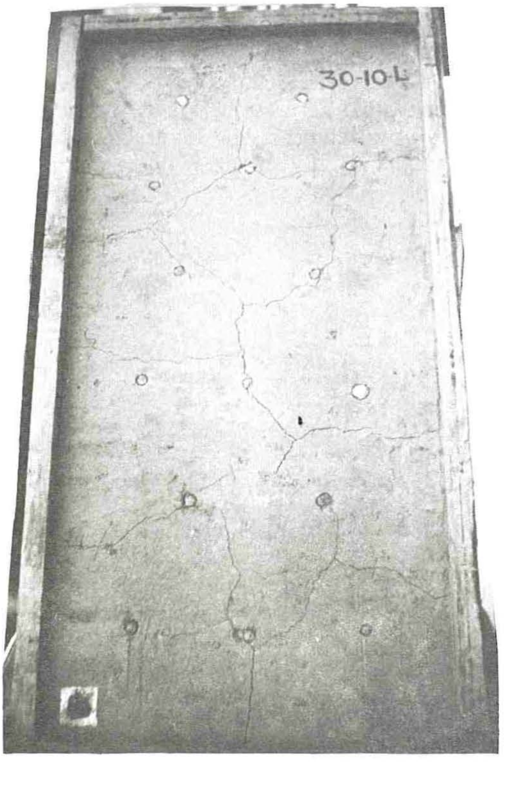

Figure 9. Crack pattern in gravel-embedded model for soil M30-10 (location of gravel shown by circles).

In accordance with the theory, cracks began to appear radially from the gravel inclusion. After further shrinkage, however, cracks radiating from adjacent inclusions became interconnected, resulting in the crack pattern shown in Figure 9. The presence of gravel substantially increased the cracking. For example, as given in Table 5, the crack intensity in a model with 15 pieces of gravel is 50 percent more than that in a control model containing no gravel.

A few qualitative conclusions pertaining to the effect of gravel can be drawn from the studies of Daniel and Durelli (3) and Shelly and Yuan (28). For instance, the higher the drying shrinkage is, the higher will be the stress concentration with gravel inclusions. Although the stress distribution around a single inclusion is independent of size, it can become significant when a group of inclusions is considered. The stress level is also modified by the spacing of inclusions; the smaller the spacing is, the larger will be the hoop stress evaluated at some point between the inclusions.

INFLUENCE OF ADDITIVES ON CRACKING

In the preceding part of this report, a number of factors regulating shrinkage cracking of stabilized pavement systems have been evaluated. Cracking can best be controlled, however, by minimizing shrinkage of the treated soil. The effectiveness of additives and treatments in reducing cracking is brought into focus in the following discussion.

Effect of Lime

In soil-cement blends, lime replaced an equal amount of cement. As given in Table 5, 3 percent lime in K03-6 and K36-6 and 4 percent lime in M30-10 reduced the crack intensity by 60, 65, and 25 percent respectively. This reduction is in excellent agreement with the shrinkage results reported by George (7). The addition of a small proportion of lime reduced the shrinkage in several soils by 30 to 40 percent. The fact that lime flocculates clay more effectively than cement and facilitates better compaction is another point in favor of recommending lime in cement-treated soils, especially in fine-grained soils. In proportioning the lime, it should be kept in mind that the compressive strength of a lime-soil-cement mixture is slightly less than that of a soil-cement mixture of equivalent cement content.

Effect of Lime and Sugar

It is generally agreed that early setting of cement is detrimental (9); therefore, it may be desirable to use slow-set cement in soil-cement work. Zube et al. (33) have reported that Type II cement, because of its slow set rate, is better than Type I cement in preventing block cracking. Orchard (21) and Arman (1) advocate the use of retarding admixtures in soil-cement.

The author of this study experimented with a trace amount of sugar in soil-cement-lime mixtures, and the crack intensity result is given in Table 5. The response of the 2 granular soils tested is remarkable. Even with 2 percent lime and $^3/_8$ percent sugar, crack intensities in K03-6 and K36-6 were reduced by 67 and 65 percent respectively. As little sugar as $^3/_{16}$ percent has been found to be effective in K03.

The reduction of cracks in sugar-treated models can be attributed to the reduced shrinkage rate of the material. As recorded by the strain gages, the shrinkage rate in

TABLE 5
EFFECT OF ADDITIVES ON CRACK INTENSITY

Additive	Additive Concentration (percent)	Crack Intensity (in.^{2}/in.^{2} × 10^{-4}) Soil K03	Soil K36	Soil M30
Gravel	Cement 10; $^1/_2$ to $^3/_4$ in. nominal size gravel			33.50
	Cement, 10 (control)			21.52
Lime	Cement, 6; lime, 3	3.33	2.29	
	Cement, 10 (control)	8.33	6.66	
Lime	Cement, 10; lime, 4			14.43
	Cement, 15 (control)			19.26
Lime and sugar	Cement, 6; lime, 2; sugar, $^3/_8$	2.74	3.44	
	Cement, 10 (control)	8.33	6.66	
Expansive cement	Cement, 0; expansive cement, 10	0.38		22.36
	Cement, 5; expansive cement, 5	0.52		22.85
	Cement, 10 (control)	8.33		21.52
Sodium metasilicate	Cement, 6; 5 percent sodium metasilicate solution applied $^3/_{10}$ gal/sq yd	2.50		
	Cement, 6 (control)	10.34		

sugar-treated models was reduced on the order of 40 percent from that in the untreated models. Previous studies (10, 27) have shown conclusively that shrinkage stresses and, thereby, cracks can be reduced by decreasing shrinkage rate.

When the compressive strengths were compared, it was found that, if ³⁄₈ percent sugar is added, the 7-day compressive strengths in K03-6 and K36-6 were reduced by 80 and 77 percent respectively. It is significant, however, that sugar-treated mixtures, if adequately cured, will develop strengths nearly equal to their untreated counterparts.

Effect of Expansive Cement

The primary use of expansive cement in concrete or soil-cement is to expand and compensate for the shrinkage that occurs during drying. Besides compensating for shrinkage, the mechanical behavior of a pavement base, which normally is continuous or restrained at the bottom or both, can be significantly modified by the expansion. If the cement base is restrained while the expansive soil-cement is curing and tending to expand, a compressive stress can be built up within the soil-cement. The experimental finding of Nagataki (20) supports this hypothesis. Upon drying, the soil-cement, which would shrink without the prior restraint, would be relieved of the compressive stress developed during the curing period. In other words, if the base material is prestressed, ultimate tensile capacity is increased by the same order of magnitude, thereby eliminating most of the shrinkage cracks.

The crack intensity in the 2 soils with varying amounts of expansive cement (expansive cement replaced an equal amount of portland cement) is given in Table 5. The results indicate that with 10 percent expansive cement in K03 (typical of a coarse-grained soil) the crack intensity is reduced by 90 percent. In fine-grained montmorillonite soil, M30, however, the expansive cement is not effective. This finding agrees with the reported results (7) in that, by replacing 50 percent of the portland cement with an equal amount of expansive cement, the shrinkage was reduced in 5 out of 7 soils tested; all 5 soils were coarse-grained soils. The test results, although limited, indicate that shrinkage can be substantially reduced by replacing approximately 50 percent of the portland cement by expansive cement. Fine-grained soils are not generally responsive to expansive cement.

Surface Hardening of Soil-Cement

As shown in Figure 3b, the shrinkage stress is highly localized; as a result, cracks originate on the exposed surface. A simple expedient to control surface cracking, therefore, would be to increase the hardness of the upper crust. Some early work on surface hardening of stabilized soil has been reported by Handy et al. (14). They showed that only a 5 percent solution of sodium metasilicate applied in the amount of ½ gal/sq yd approximately doubled the bearing strength. In this investigation 5 percent sodium metasilicate solution was sprayed in 2 equal installments at the rate of 0.3 gal/sq yd. The result was striking in that the crack intensity in K03-6 was reduced by 66 percent.

The reduction in crack intensity can be attributed primarily to 2 factors. First, because the silicate treatment boosts the surface hardness, as critical stresses occur at the exposed surface, the top-reinforced base is especially resistant to surface cracking. Second, by virtue of its ability to diffuse uniformly through the pore fluid (18), sodium metasilicate lowers the diffusivity coefficient, k, and thereby the evaporation rate. (k is a measure of the amount of moisture moving through a unit volume of the material in unit time.) The results of this study as well as those of a previous study (10) showed that by lowering the evaporation rate the incidence of cracking can be minimized.

Effect of Moisture and Density

Results concerning the effect of moisture and density on shrinkage have been reported by the author (9). They are as follows:

1. Compaction at wet or optimum moisture content results in appreciably higher total shrinkage, and molding moisture appears to have the most influence on shrinkage; and

2. Shrinkage can be reduced by improving compaction.

Because the model design was based on a dimensional analysis of the linear problem, the effect of moisture content on crack intensity in the prototype pavement cannot be predicted from model studies. Some approximate calculations, however, show that a 1 percent increase in moisture content in the field may result in as much as a 15 to 25 percent increase in the crack intensity. The writer emphasizes, therefore, that of all the factors molding moisture has the greatest influence on crack intensity. In the interest of brevity, detailed results of this study are not reported here. It has been demonstrated, however, that the maximum density attainable should be specified in the field in an effort to substantially reduce the crack intensity.

SUMMARY AND CONCLUSIONS

Employing linear viscoelastic theory, the author has presented expressions whereby stresses in base slabs subject to ambient moisture gradient can be computed. As a result of this analysis, the following results seem warranted:

1. Theoretically, shrinkage stress is highly localized on the exposed surface and decreases sharply with depth;
2. The tensile shrinkage stress on the exposed surface of a soil-cement slab attains maximum value during the early stages of drying (40 to 100 hours depending on the restraint condition) and then decreases rapidly; and
3. The maximum shrinkage stress, which typically varies from 0.2 to 0.3 $E_m S_\infty$, is much greater than the tensile strength of soil-cement in normal use; consequently, the surface of the slab will crack or flow under stress.

Application of these results would suggest that, if cracking is minimized, a soil-cement base warrants special attention and curing during the first critical 2 to 4 days.

The shrinkage cracking of soil-cement base was investigated by using a model whose design was based on a dimensional analysis of the linear problem of shrinkage cracking. The results are as follows:

1. The crack intensity, i (defined as area of cracks per unit area), decreases with an increase in the thickness of the slab, cement content, and the coefficient of subgrade friction;
2. The crack intensity decreases with a decrease in the modulus of viscosity of the soil and the shrinkage rate;
3. The crack intensity increases with the type and amount of clay-sized particles in the soil (clay content exerts more influence on cracking than does any other factor); and
4. Large aggregates (nominal size ½ in. and larger), by virtue of their ability to intensify the stress in the shrinking matrix, enhance crack intensity.

As a result of this study, it is recommedned that cement equal to or slightly in excess of that required to meet the PCA-ASTM criterion be used.

The search for treatments to reduce cracking led to several promising additives. Lime and lime with a trace amount of sugar proved to be the best in a variety of soils. Expansive cement admixture and sodium silicate surface treatment are effective in well-graded, coarse-grained soils. The effect of moisture content on shrinkage cracking is sufficient to warrant a special effort to compact the soil-cement at or below, but never above, the optimum moisture. The model studies confirm that shrinkage cracking can be reduced by improving compaction.

NOTATION

The following notations were used in this paper.

D = flexural rigidity;
E = Young's modulus;
E_K = Young's modulus in Kelvin model;
E_m = Young's modulus in Maxwell model;
f = surface factor;

G_K = shear modulus in Kelvin model;
G_m = shear modulus in Maxwell model;
h = thickness of the slab;
k = diffusivity coefficient of shrinkage;
K = bulk modulus;
B = width of slab in x-direction;
S = free, unrestrained linear shrinkage;
S_{av} = average shortening per unit length;
S_∞ = final shrinkage, value of S when $t = \alpha$;
t = time;
u, v, w = displacements in x-, y-, z-directions;
η_K = modulus of viscosity in Kelvin model;
η_m = modulus of viscosity in Maxwell model;
ν = Poisson's ratio;
θ = moisture content by weight;
σ_{xx}, σ_{yy} = normal components of stress parallel to x- and y-axes;
ϵ_{xx}, ϵ_{yy} = stress-induced unit elongations in x- and y-directions;
K03-06 = soil 3 with 6 percent cement, predominant clay mineral kaolin;
K36-06 = soil 36 with 6 percent cement, predominant clay mineral kaolin; and
M30-10 = soil 30 with 10 percent cement, predominant clay mineral montmorillonite.

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Discussion

J. B. METCALF, Australian Road Research Board, Victoria—The author has produced a most interesting analysis of cracking in soil cement slabs. However, I do not believe the problem he has tackled to be the central problem in stabilization. In taking the problem of cracking density, he produces arguments that show reduced cracking density with higher cement contents, i.e., with higher strengths. Yet, this is the condition that leads to the worst form of cracking, and I submit that the form of cracking is more important than its intensity as suggested here. The most common cause of failure due to cracking is the appearance of widely spaced, wide cracks in cement stabilized pavements. For this condition, the cracking density may be quite low and the strength quite high, and this is to be avoided. The first question for George is, Can

he reorient his approach to produce estimates of crack width and crack spacing rather than cracking intensity and point the direction in which cement content and so on should be moved to minimize this. I would expect to find the reverse of his present conclusions. The second question is of a rather more detailed nature: Can he provide information on the proportion of cracking due to cement hydration, rather than to drying out, because good construction practice would lead to a minimum of drying out?

K. P. GEORGE, Closure—The writer appreciates the interesting comments presented by Metcalf. Despite the apparent disagreements, the writer finds that the comments are essentially the same as those presented in the paper.

The answer to Metcalf's first question may be seen in the paper in the section entitled "Effect of Cement Content (Tensile Strength) on Crack Intensity." Should the shrinkage data, similar to those shown in Figure 8 of the paper, be the criterion, the writer tends to agree with Metcalf's hypothetical conclusions. Nevertheless, the model study shows that, for treatment levels at or slightly above the PCA-ASTM criterion, the crack spacing is increased and the crack width is decreased (in Metcalf's terminology, widely spaced, narrow cracks). The writer, therefore, would like to reaffirm his conclusion that for granular soils it may be desirable to use slightly more cement than is required to meet the PCA-ASTM criterion.

For an answer to the second question raised by the reviewer, it would be pertinent to refer to the results given in another report (9). In that report, the writer showed that the cement in soil-cement takes up moisture to result in self-desiccation and shrinkage of the order of 17 percent of the maximum shrinkage. A conclusion, based on this result, would be that perhaps 15 to 20 percent of the cracking results because of cement hydration.

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